Analysis of Simultaneous Operational Failure of Critical Facilities due to Earthquake, for a California Utility

Prepared by
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EXECUTIVE SUMMARY

This study presents an estimate of the probability that a single earthquake could cause simultaneous operational failure of geographically disperse data centers operated by a California utility. Three facilities are considered: a grid control facility (denoted herein by GCF), a data processing facility (DPF), and a backup data facility (BDF) that can perform the functions of either GCF or DPF, should either be rendered inoperative. This study estimates two probabilities: (1) that within the next 5 years a single earthquake could render both the grid control and backup facilities inoperative; and (2) that within the next 5 years a single earthquake could render both the data processing and backup facilities inoperative.

The work was performed by researchers at the California Institute of Technology in Pasadena, CA, in collaboration with researchers at the United States Geological Survey in Pasadena, CA and Golden, CO. Caltech designed and directed the research, examined the seismic vulnerability of the three sites, and quantified the two probabilities desired. The USGS performed the hazard analysis.

Hazard analysis. The USGS created a database of 250,000 earthquake scenarios that could affect the three sites. The database includes information about the probability distribution of shaking at each site under each scenario earthquake. The database uses the same authoritative seismological and other mathematical information as the USGS 2002 National Seismic Hazard Maps. Important outcomes of the hazard analysis are that:

(1) The most severe earthquake scenarios for the primary facilities and for the backup facility are, respectively, the Puente Hills and San Joaquin Hills faults. The largest earthquakes on these faults could cause potentially damaging shaking at all three facilities, with median peak ground accelerations shown in Table E-1. For reference, 0.05g is commonly considered the threshold of damaging *PGA*, and shaking in the epicentral region of the Northridge earthquake was on the order of *PGA* = 0.9g.

Table E-1: Median shaking intensities in most severe earthquake scenarios

Facility	Median* *PGA* in scenario earthquake	
	M7.4 Puente Hills	M7 San Joaquin Hills
Data processing facility	0.92g	0.15g
Grid control facility	0.82g	0.12g
Backup data facility	0.23g	0.65g

* These are best-estimate shaking intensities; they could be 60% higher or more.

(2) The siting of the backup facility is effective at reducing the probability of simultaneous failure: because the backup is located far from the primary facilities, even given the most serious earthquake (the Puente Hills event), it is unlikely that shaking in excess of approximately 0.2g will occur at *both* the primary and backup facility.

Vulnerability analysis. The utility has taken extensive action to control the seismic vulnerability of its facilities. In a detailed examination of the equipment at each facility, we found that almost all equipment appear to have been seismically restrained, and that there is extensive redundancy of building service equipment, including multiple sources of uninterruptible power supply (UPS), emergency generators (and in one case multiple parallel emergency generators), and air conditioning (crucial to computer operations). Corporate real estate personnel and building engineers informed us of ongoing efforts to enhance the seismic resistance of equipment, most notably the seismic bracing of raised access flooring, an expensive and time-consuming effort that is being performed over the space of several years. In addition to concerns about the remaining unbraced raised access flooring, there are a few remaining conditions that may represent important weak links, where remediating them might substantially reduce failure probability.

1. Some critical computer equipment (tape silos, PC system units and monitors), some consoles in which critical computers are installed, and all cubicles in operationally critical areas appeared to lack seismic restraint.
2. Long unbraced runs of sprinkler pipe were observed in some critical areas, and in many places there appeared to be potential for interaction between sprinkler pipe and ventilation ducts. In some of these areas, sprinklers are wet systems, meaning that damage to pipes could release water on sensitive electrical equipment below.
3. Although emergency generators appeared to be seismically restrained, some critical components were not, such as day tanks, fuel pumps, and batteries used for starter motors. Failure of these unrestrained components could render the emergency generators inoperative.
4. The data processing facility has cooling towers that lose water to evaporation. There is no on-site water supply in case of service interruption from the local utility.

Risk analysis. The probability that both the data processing facility and the backup facility will fail in a single earthquake in the next five years is estimated to be 0.4%. The

probability that both the grid control facility and the backup facility will fail in a single earthquake in the next five years is estimated to be 0.05%. By remediating all items notes in the vulnerability analysis as "possible weak links," the probability of either failure is estimated to be less than 1 in 100,000. Details are shown in Table 1.

Table 1. Risk analysis results: estimated failure probability within the next 5 years

Conditions	Data processing facility	Backup data facility	Grid control facility	Data processing and backup	Grid control and backup
Current	0.055	0.032	0.008	0.004	0.0005
Remediate all	0.002	0.001	0.003	$< 1 \times 10^{-5}$	$< 1 \times 10^{-5}$

Recommendations. We recommend remediating the few remaining unanchored items, and performing mechanical testing of existing anchorage (if such tests have not already been done). It may not be possible to make some of the unanchored items seismically rugged, particularly the tape silos, which we understand are very sensitive. Perhaps these could be placed in distant location without severe impact to their operation and maintenance.

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AHU	Air handling unit
ATC	Applied Technology Council
ASCE	American Society of Civil Engineers
BDF	Backup data facility
CPU	Central processing unit
CRT	Cathode ray tube
DDC	Direct digital control
DPF	Data processing facility
FEMA	Federal Emergency Management Agency
g	Acceleration due to gravity
GCF	Grid control facility
ITOC	Information Technology Operations Center
kW	kilowatt
LCD	Liquid crystal display
OpenSHA	Open-source seismic hazard analysis software developed by SCEC and USGS
PGA	Peak ground acceleration
SCEC	Southern California Earthquake Consortium
UPS	Uninterruptible power supply
USGS	United States Geological Survey
V	Volt

1 INTRODUCTION

1.1 OBJECTIVES

This section describes the principal objectives of the project. A California utility company is interested in understanding whether three data centers located within 100 miles of each other share a common seismic risk that could result in data processing or grid control being unavailable after a single earthquake. Let the facilities be denoted by GCF (grid control facility), DPF (data processing facility), and BDF (backup data facility). The analysis consists of data collection, hazard analysis, and quantitative risk assessment. In particular, it is desirable to estimate:

- The probability that both the primary and backup data processing facilities (DPF and BDF) would be simultaneously rendered inoperative during a planning period of 5 years (i.e., between August 1, 2006 and July 30, 2011), because of a single earthquake.
- The probability that both the primary and backup grid control facilities (GCF and BDF) would be simultaneously rendered inoperative during the same planning period.

By “rendered inoperative” is meant that the facility is largely incapable of performing critical grid control or other data processing functions for a period of more than a few minutes, either because

- the building is rendered unsafe to enter or occupy, i.e., red-tagged by local building department officials, or because
- operational failure occurs to critical equipment systems: computers, mechanical, electrical, or plumbing components, to certain architectural components such as raised access floors, or to off-site utility services such as nearby electric substations. Such operational failure could occur because of shaking, earthquake sprinkler leakage, operator error, and possibly other reasons.

We consider here only earthquake shaking and earthquake sprinkler leakage as causes of operational failure, so to that extent the risk analysis may underestimate the total probability of operational failure. (Other hazards include landslide, liquefaction, faulting, inundation,

fire, hazardous material spills, including hazmat from neighbors or nearby transportation corridors.) We consider in an approximate way the failure of off-site utility services.

1.2 PROJECT PARTICIPANTS

Staff of the California Institute of Technology (Caltech) led the project, developed the methodology, performed the facility data collection and systems analyses, performed the seismic risk analysis, and produced this report. The United States Geological Survey (USGS) performed the seismic hazard analysis. They identified possible future earthquakes that could occur and affect the sites and estimated the shaking intensity in each event and the frequency with which each event is expected to occur.

1.3 APPROACH

The analysis has three part: hazard analysis, vulnerability analysis, and risk analysis. In the hazard analysis, one estimates the frequency and severity of events that could affect the sites. In the vulnerability analysis, one estimates the consequences of an event, given its severity. In the risk analysis, one combines the results of the hazard analysis (what is the frequency of damaging events) and the vulnerability analysis (what are the consequences given an event's severity) to estimate the resulting frequency with which various consequences occur. These steps are now briefly summarized.

1.4 ORGANIZATION OF THE REPORT

Section 1 (this section) has presented the study objectives, briefly identified the project participants, and summarized the approach. Section 2 summarizes the hazard analysis, i.e., it deals with the earthquakes and site shaking at each site. Section 3 presents the vulnerability analysis, i.e., the relationship between shaking intensity and loss of function. Section 4 presents the risk analysis, i.e., considering (a) how frequently the earthquakes occur and (b) the loss of function given each earthquake, what is the likelihood within a given planning period (here, 5 years) that loss of function will occur. Section 5 summarizes our findings. Section 6 presents complete bibliographic references for sources cited here.

Appendices A, B, and C contain fault trees developed to depict the events that could cause failure of the facilities. Appendix D summarizes defensibility and state-of-the-art aspects of this study. Appendix E lists limitations of the study.

2 HAZARD ANALYSIS

2.1 HAZARD ANALYSIS METHODOLOGY

2.1.1 Purpose of the hazard analysis

The purpose of the hazard analysis is (1) to identify the earthquakes that could affect each site, (2) to estimate the intensity of shaking at each site from each earthquake, and (3) to estimate the probability that the earthquake would occur. (More precisely, we estimate the mean rate at which each of a large number of possible earthquake scenarios occur, and the probability distribution of shaking at each site.) Other seismic hazards are considered qualitatively.

2.1.2 Probabilistic seismic hazard analysis

While the technical details of the hazard analysis are quite complicated, a simple summary can be provided. Seismic hazard derives from the possible future rupture of earthquake faults. The fault rupture generates waves of motion that propagate through the earth to the surface, and result in shaking of the ground. The intensity of shaking depends on the earthquake magnitude, the direction and distance from the fault rupture to the site, and the stiffness of the soil near the ground surface at the site (more precisely for the technical reader, the average shearwave velocity in the upper 30 meters of soil, denoted here by Vs30), and certain other features of the fault rupture. In general, a larger magnitude, closer distance, and softer soil each act to increase the shaking at the site.

Earthquake shaking at a given site is measured with a variety of ways, most commonly the peak ground acceleration (*PGA*), the maximum rate at which the velocity of the ground changes during an earthquake, measured in units of gravity. We used three measures of shaking intensity: peak ground acceleration, and two, additional standard measures that will not be defined here but will be noted for the technical reader: pseudo-acceleration spectral response at 0.2 second and 1.0 second periods, with 5% viscous damping. It is of interest here to estimate intensity by each measure at each of three sites.

To do this, the following procedure is used: first, a map of the faults in the region is created, and the rate at which each fault produces earthquakes of various magnitudes is estimated. These comprise an earthquake rupture forecast; we used the same one employed

by the US Geological Survey to create the 2002 National Seismic Hazard Maps (detailed in Frankel et al. 2002).

The analysis begins by selecting a given fault, a given magnitude, and a particular segment of the fault that must rupture to produce the given magnitude. Earthquake faults can produce a continuous range of magnitude, but for convenience we chose discrete values in 0.05-magnitude increments. Each fault can produce a given magnitude earthquake along any portion of its length. That is, except in the largest events, only a portion of the fault actual ruptures. We discretized rupture locations in 5-km steps. The combination of fault, magnitude, and fault rupture segment is sometimes referred to as a scenario earthquake.

Next, one uses a mathematical relationship between magnitude, distance from the rupture to the site, site soil characteristics, and shaking intensity to estimate the probability distribution of shaking at the site, given that scenario earthquake. The relationship is commonly called a seismic attenuation relationship, and is typically derived by fitting a smooth curve to a plot of data points from observations in past earthquakes. The attenuation relationship typically provides both a median value of shaking intensity and one or more measures of uncertainty. Here, several attenuation relationships are used, the same ones employed for the 2002 National Seismic Hazard Maps (namely, Atkinson and Silva 1997, Boore et al. 1997, Campbell and Bozorgnia 2003, and Sadigh et al. 1997). Median shaking intensity and uncertainty in shaking intensity are calculated for each site of interest for each scenario earthquake and each attenuation relationship.

Furthermore, two measures of uncertainty are used: one to depict the variability of the *median* value of ground motion (which can vary between earthquakes, and be higher or lower over the entire affected area than the average earthquake—this is sometimes called the interevent variability), and one to depict the remaining uncertainty in ground motion (i.e., how shaking intensity can vary from site to site, in a given earthquake—this is called the intraevent variability). We used a standard source, Lee et al. (2000), to distinguish inter- and intraevent variability.

Site soil characteristics can be found by reference to a soil map and checked to the extent possible with geotechnical reports for the facilities. The soil map used here was created in 2000 by the California Geological Survey (Wills et al. 2000). Each site was located on the map using its latitude and longitude, determined from its street address using Microsoft

Streets and Trips (Microsoft Corp., 2004) and checked using Google Earth Plus (Google Inc., 2005).

To recap, the product of the seismic hazard analysis is a database containing estimates of the median and two measures of variability of shaking severity for each site, each fault, each magnitude, and each rupture location. The faults and rupture locations considered here were all those contained in a recent, authoritative USGS fault model that are located within 200 km of any of the three sites of interest. Approximately 3,000 faults and other sources of earthquakes (called background sources, where earthquakes are know to have occurred but no fault is known to exist) are examined. Considering magnitudes and rupture locations, approximately 62,000 scenario earthquakes are considered. After multiplying by 4 attenuation relationships (also authoritative, having been selected by the USGS for its recent National Seismic Hazard Maps) and 3 levels of inter-event variability[1], we calculated approximately 750,000 scenarios, and for each of three sites, three measures of shaking intensity, each with a median value, an estimate of intra-event variability, and a mean annual rate of occurrence.

2.2 HAZARD ANALYSIS RESULTS

The database described in the previous section was compiled. While the real importance of the database lies in its implications for operational failure probability, some aspects of the hazard data are interesting by themselves. We can identify the earthquake faults that produce the strongest shaking at each site.

In terms of median peak ground acceleration (*PGA*), among all earthquake sources considered here, the newly identified Puente Hills blind thrust fault appears to be capable of producing the strongest shaking at both DPF and GCF. The largest magnitude event on this fault (M7.4) is estimated to produce (median) *PGA* at DPF and GCF of 0.92 and 0.82g, respectively. (Considering uncertainty, shaking could be as much as double this level of shaking with 10% probability. The technical reader should note that the attenuation relationship producing these levels of shaking is Boore et al. 1997, and that the 90th percentile refers to an arbitrary direction of motion, as discussed by Baker and Cornell 2006.) Even the median *PGA* represents very strong shaking, as strong as the most strongly shaken area of the 1994 Northridge earthquake. In the same event, the median *PGA* at BDF is estimated to be

[1] For the technical reader, the 3 levels of inter-event variability span are Gauss points.

0.2g. Although but far less severe than at DPF and GCF, this is potentially damaging shaking, approximately equal to that experienced at the Los Angeles City Hall in the Northridge earthquake. While we have yet to estimate the probability of operational failure at BDF in such an earthquake, 0.2g is well above the level considered negligible.

The event that is estimated to produce the strongest shaking at BDF is a M7 event on the San Joaquin Hills Thrust fault, with median *PGA* at BDF equal to almost 0.7g, and median *PGA* at DPF and GCF of 0.15g and 0.12g, respectively. Again, considering uncertainty, *PGA*s in such an earthquake could be substantially greater. A map showing shaking intensities in both earthquakes is shown in Figure 2-1.

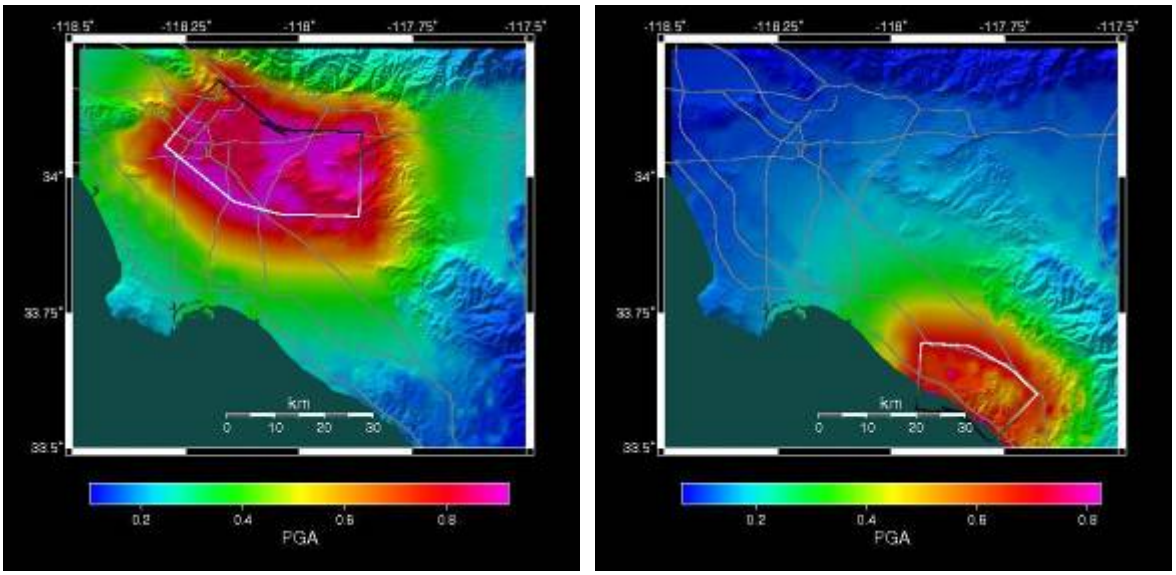

Figure 2-1. Scenario earthquakes producing the strongest shaking at the facilities studied here. The left-hand map shows shaking intensity in the events in a M7.4 Puente Hills blind thrust earthquake. The right-hand map shows shaking intensity in a M7 San Joaquin Hills thrust earthquake. The maps show the median estimated peak ground acceleration for this event (i.e., shaking with 50% exceedance probablity given the occurrence of this fault rupture). The boxes near the area of strongest shaking indicate the boundaries of the fault rupture. Facility locations are omitted for confidentiality.

Considering all earthquakes that could affect these sites, we can estimate the frequency with which earthquakes occurs occur that cause *PGA* > 0.05 times the acceleration due to gravity (an approximate threshold for potentially damaging shaking). See Table 2-1. The third column in the table is simply 1 divided by the second column; it shows the average number of years between events causing *PGA* > 0.05g. Because the backup facility (BDF) provides a backup for both DPF and GCF, it is of interest to know how frequently earthquakes occur causing *PGA* > 0.05g at *both* DPF and BDF, or at *both* GCF and BDF. See Table 2-2. (For the technical reader, these frequencies reflect the sum of mean occurrence

rates of all scenario earthquakes where the median *PGA* > 0.05. Each of the four attenuation relationships is weighted equally.)

Table 2-1. Average frequency of potentially damaging earthquakes

Facility	Average frequency of *PGA* > 0.05*g* (times per year)	Equivalent average return period (years)
Data processing facility	0.14	7.3
Backup data facility	0.13	7.9
Grid control facility	0.12	8.2

Table 2-2. Average frequency of potentially damaging earthquakes affecting main and backup site

Facilities	Average frequency of *PGA* > 0.05*g* (times per year)	Equivalent average return period (years)
Data processing and backup	0.082	12
Grid control and backup	0.071	14

Figure 2-2 shows the mean frequency with which single earthquakes occur causing median *PGA* to exceed *x* times gravity at *both* the data processing facility *and* the backup facility, as a function of *x*. The figure shows the same information for shaking at both the grid control facility and the backup facility. Note that no scenario earthquakes are estimated to produce a median *PGA* > 0.25g at both DPF and BDF, or at both GCF and BDF. (The technical reader should note the word "median" in the previous sentence. It speaks to the *PGA* produced by each scenario earthquake with 50% probability, not with complete certainty, so there is some nonzero probability of *PGA* > 0.25g at two of the facilities.)

Because earthquakes are estimated to produce shaking at each *individual* facility far more frequently than shown in Figure 2-2, one can observe an important success in siting the backup facility. Because of where the backup is located, far from each primary facility, it is unlikely that shaking in excess of 0.25g will occur at *both* the primary and backup facility even given the most serious earthquake examined here.

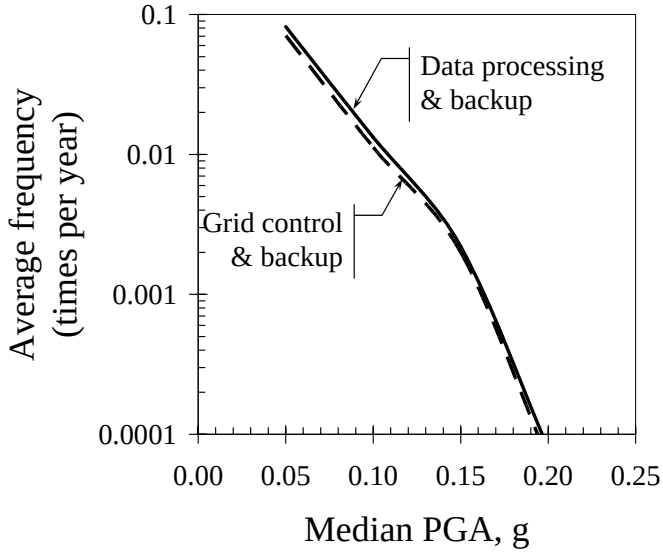

Figure 2-2. Two-site PGA hazard curves

2.3 OTHER HAZARDS

Other common seismic hazards include ground failure such as liquefaction, lateral spreading, and landslide; fault rupture at the ground surface directly beneath the building; fire following earthquake; and inundation due to dam failure. These hazards are considered here only briefly and qualitatively. We examined geotechnical reports for the subject facilities that were provided by the utility and found no indication that the geotechnical engineer considered liquefaction, lateral spreading, or landslide likely to occur. The sites are not located atop any known fault traces. Post-earthquake conflagration is a possibility in a large earthquake, more so for the grid control and data processing facilities than for the backup data facility, which has the advantage of being presently located in a largely open area near a fire station. All three facilities have concrete cladding, noncombustible roofs, and few window openings, all of which mitigate the potential for fire to spread from adjacent buildings. We have no information in our files about the potential for inundation due to dam failure; the utility should consider examine that possibility.

3 VULNERABILITY ANALYSIS

3.1 VULNERABILITY ANALYSIS METHODOLOGY

3.1.1 Purpose of the vulnerability analysis

The purpose of the vulnerability analysis is to relate shaking intensity to its consequences at for a particular facility. In the present case, two consequences are of interest: red-tagging and loss of function due to nonstructural causes (equipment damage, etc.). Both are treated as "binary" events, meaning they are depicted as either occurring or not occurring, and nothing in between. (In fact, there are varying degrees of loss of function for these facilities, but the present analysis simplifies loss of function to make the analysis practical within the available time.) Both are treated probabilistically, meaning that given each of the scenario earthquakes, we desire to estimate the probability of red-tagging, loss of use, or both.

3.1.2 Fault trees and fragility functions

To calculate the probability of loss of use, we employ a methodology called fault-tree analysis. It was first used in connection to estimate the safety of the Minuteman Missile launch control system in 1962. It has since been used to assess the reliability of various complex systems. A fault tree is a diagram that shows, at its top, a box representing an undesirable outcome, such as "Grid control failure." (See Figure 3-1.) The possible causes of the top event are depicted below in a row of events that could cause the top event, such as "red tagging" or "equipment failure." These lower events are connected to the top event by lines and a logic symbol that indicates whether *any* of the lower events causes the top event (an "or" gate, i.e., the top event occurs if the first or the second or the third or any combination of the lower events occurs) or whether *all* of the lower events must occur to cause the to event (an "and" gate). There are other gates that can be used in fault-tree analysis, but only these two are used and explained here. Each lower event can be further broken down into even-lower events connected to the upper ones by logic gates, until the probability of all the lowest events (called basic events) can be estimated. Here, the basic events are red-tagging of the building and failure of various building and equipment components. The probabilities of the basic events must be estimated as a function of shaking intensity. See Appendices B, C, and D for fault trees for the facilities examined here.

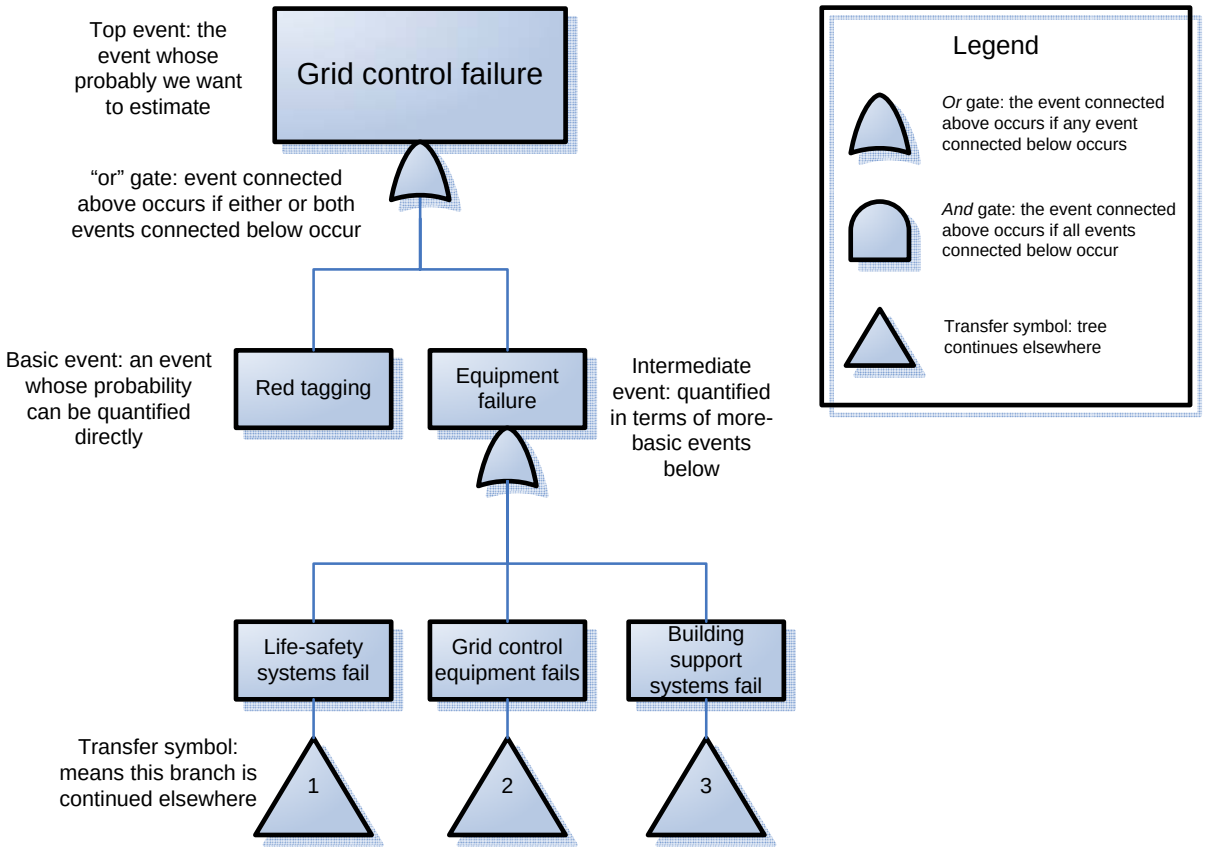

Figure 3-1. Illustrative portion of a fault tree. The event at the top of the tree is the one whose probability is desired. Logic gates equate the occurrence of an upper event with events below it. The diagram indicates that grid control fails if either the building is red-tagged (i.e., declared by a building department official to be unsafe to enter or occupy) or the equipment fails. Equipment failure is further broken down into lower events. The triangles at the bottom indicate that this is only a portion of the complete tree, and that more basic events shown elsewhere contribute to the events at the bottom of the diagram.

The relationship between shaking intensity and failure probability of a component is referred to here as a fragility function. The fragility function depends on what kind of component is shaken and its seismic installation conditions. For example, Johnson et al. (1999) provide several fragility functions for battery racks in uninterruptible power supply equipment, depending on whether the battery rack is anchored to the floor, has longitudinal braces, spacers between the batteries to prevent their pounding into each other and breaking their conductors, and other relevant features. Figure 3-2 illustrates fragility functions for battery racks, using parameters provided by Johnson et al. (1999).

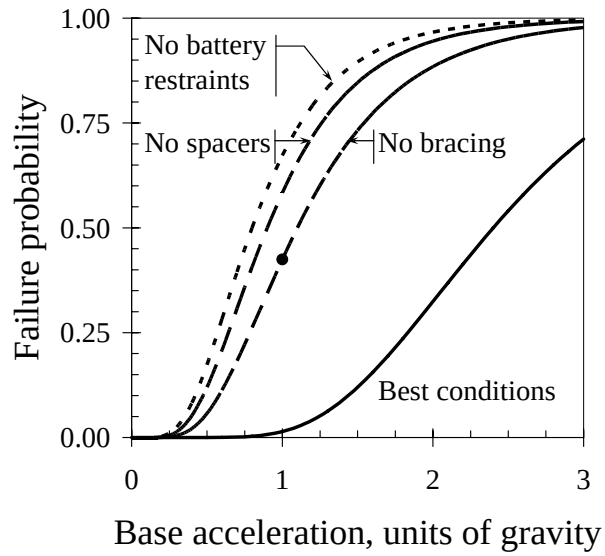

Figure 3-2. Illustrative fragility functions for UPS battery racks. They show the probability that a battery rack will fail to operate after its base experiences various levels of acceleration. For example, if the battery rack lacks longitudinal cross bracing and experiences shaking of 1g (the acceleration due to gravity), there is an approximately 40% probability that it will be damaged to the extent that it will fail to operate after the earthquake (this is the point highlighted by the large black dot).

To analyze the fault tree, one creates a mathematic equation for the probability of the top event as a function of the probabilities of the basic events. A complete treatment of the math is omitted here. Suffice it to say that the probability that event *i* occurs (denoted by P_i) if lower events 0, 1, 2, … and *i*-1 all occur is the product of the probabilities of the lower events (denoted by P_0, P_1, etc.):

$$\textit{And}\text{ gate: } P_i = P_0 \times P_1 \times P_2 \ldots \times P_{i-1} \tag{1}$$

while the probability that event *i* occurs if any one of the lower events occurs is given by

$$\textit{Or}\text{ gate: } P_i = 1-(1-P_0)\times(1-P_1)\times(1-P_2)\times\ldots\times(1-P_{i-1}) \tag{2}$$

(These equations ignore correlation for the sake of simplicity.) The equations can be combined for any tree comprised of *and* and *or* gates, resulting in a single equation for the occurrence of the top event as a function of the probabilities of the basic events. In the case of the fault tree shown in Figure 3-1, if we had the failure probabilities for each of the lowest four events (red-tagging, life-safety system failure, grid-control equipment fails, and building support system fail), we could estimate the probability that grid control fails as

$$P_4 = 1-(1-P_0)\times(1-P_1)\times(1-P_2)\times(1-P_3) \tag{3}$$

where P_4 denotes the grid control failure probability and P_0 through P_3 denote the probability of the contributing events.

Since the probabilities of the basic events are themselves functions of shaking intensity (e.g., the base acceleration of an equipment component), one can calculate the probability of the top event as a function of shaking intensity, creating a fragility function for the top event.

The probabilities P_0, P_1, etc., could refer to the probability of equipment failure or to the probability of red-tagging. To estimate the failure probability of equipment, we draw extensively from Johnson et al. (1999), who themselves compiled research performed for the Electric Power Research Institute (EPRI) and others. To estimate the failure probability of the structure (i.e., the probability of red-tagging), we use a combination of a state-of-the-practice performance-based earthquake engineering methodology created for the Federal Emergency Management Agency called FEMA 356 (American Society of Civil Engineers, 2000), and the standard methodology for post-earthquake building safety evaluations called ATC-20 (Applied technology Council 1996). The combination is described in more detail later in this chapter.

3.2 VULNERABILITY ANALYSIS OF EQUIPMENT

Caltech personnel performed site visits site visits on June 29 and 30, 2006 and July 5, 2006 to observe and document the seismic condition of operational equipment (computers and other control equipment), and the structural and other nonstructural components on which the operational equipment depend. These observations were necessary to construct the fault trees. Several utility employees and contractors accompanied Caltech staff on each site visit.

We observed operational equipment to develop an understanding of the components on which critical equipment system depend, their number and degree of redundancy (i.e., how many duplicate components exist that can perform a single function, even if one or more is rendered inoperative), and the installation and maintenance features that tend to be most important to their failure probability. By “failure probability” is meant the likelihood that a component will fail to operate after an earthquake as a result of overturning or other damage. For example, we examined the seismic anchorage of electrical cabinets to the floor slab beneath them, because seismic anchorage is strongly related to the probability that the cabinet would overturn in the event of an earthquake.

We also observed a few structural and architectural components relevant to the performance of operational equipment, mostly looking for features that indicate the potential for interaction between critical equipment and structural or other nonstructural components. For example, flexible equipment installed very close to structural members might pound into the structural member and be damaged. Another example is the raised access flooring on which computer equipment rests; if the flooring collapses the computer equipment can be damaged and rendered inoperative.

We relied largely on the structural drawings for information about aspects of the structural system related to red-tag probability, since relevant details are typically largely concealed within the structural components themselves (for example, the steel reinforcing bars within a reinforced concrete member) or behind architectural finishes (e.g., steel connections under layers of fireproofing and gypsum wallboard)[2].

Note that throughout this report we frequently say, “appeared to lack seismic restraint” or “appeared to be anchored,” etc. We make the former kind of statement because in most cases it impossible or impractical to observe each piece of equipment from all vantage points, and to determine definitively that the no restraint was present. We make the latter kind of statement because, although we frequently observed what appeared to be anchor bolts or other restraints, we performed no mechanical tests to ensure that bolts or other anchors were actually secured to the slab or wall, or installed according to the manufacturer’s directions. There are well-known examples of the tops of bolts with nuts attached to them glued to equipment to make it appear that the bolts are properly anchored into slab or wall, but of course provide no seismic resistance.

If the utility has not yet performed such tests, e.g., just after installation, it should consider doing so to ensure construction quality.

3.2.1 Observations of equipment at grid control facility

This facility was visited on June 29, 2006. It controls electric transmission within the utility’s territory. See Appendix A for the fault tree of this facility.

Grid control room. The grid control room has a low 50-ft and 30 ft raised access floor on approximately 3-inch pedestals which appeared in at least some cases not be to secured to the

slab, in others possibly secured by mastic (Figure 3-3). Such glue is intended only to hold the pedestals during installation, not to secure them for the long term. It is thought that concrete surface dust and other defects result in low shear capacity. Mechanical anchors are preferable. All raised access floors appeared to have stringers connecting pedestals—a desirable feature. The room contains three computer consoles and a display board. We were told that one of the consoles provides a backup for either of the other two. The consoles appeared to have no positive restraint against sliding (Figure 3-4), meaning that in strong shaking they could slide or overturn. This is a potential weak link at this facility.

Some of the computer monitors and system units (the boxes that house the disk drives, system board, central processing unit, etc.) in the consoles were observed to be restrained against sliding, and some appeared to be unrestrained (Figure 3-5). Lack of restraint tends to increase the probability that the component will slide, overturn, and possibly cease to operate in the event of an earthquake. Unrestrained computers and monitors could be a weak link at this facility. The large display board contains a number of liquid crystal displays, which appeared to be well secured to a Unistrut steel frame, itself apparently anchored to the floor and wall (Figure 3-6).

Figure 3-3. Grid control room and pedestals supporting its 3-in raised access floor

[2] Exceptions exist, especially structural changes made to accommodate architectural or equipment features. A classic example is a crucial brace cut to accommodate a door. We did not observe any such obvious defects, although some may exist.

Figure 3-4. Grid control room consoles appeared to lack seismic restraint

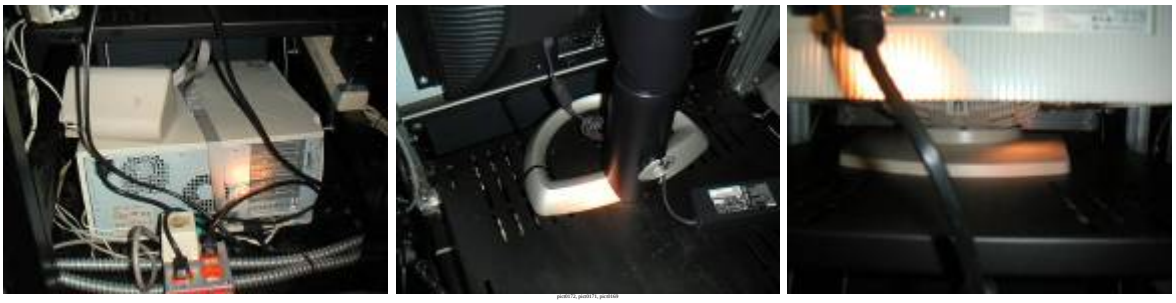

Figure 3-5. Some but not all computer equipment in grid control room appeared to be restrained

Figure 3-6. Grid display board (left), and its restraint at bottom (center photo) and top (right)

The overhead suspended ceiling system appeared to lack compression struts, although splay-brace wires were present (Figure 3-7). (Compression struts are rigid rods extending vertically from the ceiling framing members to the floor slab above, and are required by building codes for seismic resistance; the wires alone are considered inadequate to prevent ceiling collapse.) In practice, if the ceiling extends to the walls of the room, the walls tend to restrain (or “capture”) the ceiling more effectively than compression struts and splay braces, but the ceiling in this room does not extend to all the adjacent walls, and instead slopes up to the ceiling on the side of the room where one enters (i.e., opposite the display boards). Light fixtures near the point of observation had safety wires, used to inhibit the fixtures from falling if the ceiling collapses (Figure 3-8).

Figure 3-7. Suspended ceilings in grid control room.

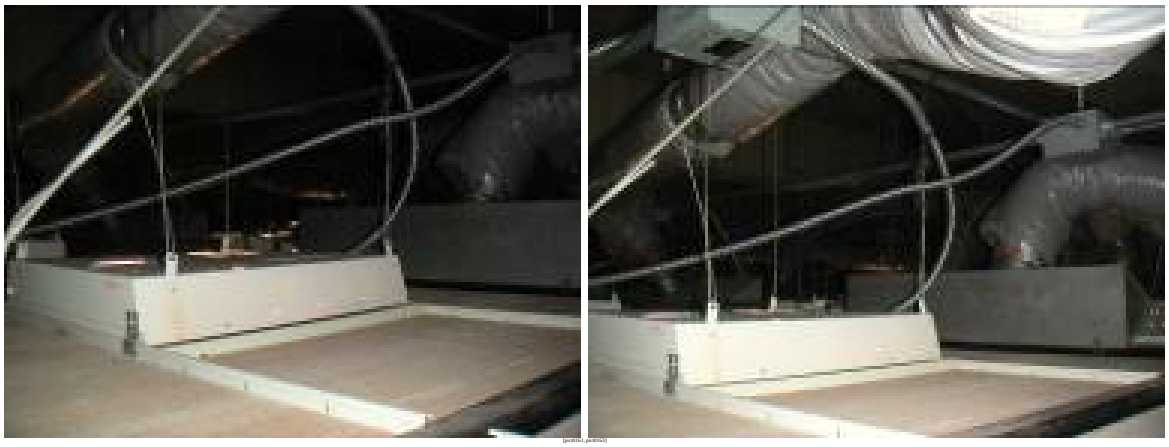

Figure 3-8. Safety wires observed on light fixtures in grid control room ceiling

Main computer room. Equipment in this room stands on a raised access floor on 12-in pedestals, the pedestals appeared to be fixed to the floor slab with mastic and mechanical anchors, and to have stringers positively attached to the pedestal heads, but to lack diagonal bracing (Figure 3-9). The computers comprise what was described to Caltech as a next-generation energy control system. Computer system units and monitors were secured to racks with fabric straps and the racks secured to the concrete floor slab by a system of diagonal metal rods, turnbuckles, and bolts anchored to the slab (Figure 3-10). The bracing for the racks was probably not designed to brace the raised access floors against excessive lateral displacement; it is unclear whether it would do so in practice. The lack of bracing for the raised access floors could be a weak link at this facility.

Main computers are largely redundant, meaning that any of two or more computers can perform the same function, if one is damaged. The suspended ceiling system was observed to have compression struts and diagonal splay braces every 6 to 12 ft, and the ceiling appeared to be captured on all four sides by walls. Some light fixtures in the main computer room

appeared to lack safety wires secured to the slab above (Figure 3-11). This is primarily a life safety hazard, although in the 1983 Coalinga earthquake, falling light fixtures caused damage to equipment below (EQE Inc., 1986). The room is protected by a Halon system for fire suppression, and the Halon tank is secured to the wall (Figure 3-12).

Figure 3-9. Anchorage of pedestals in main computer room raised access floor

Figure 3-10. Restraint of computer equipment in main computer room

Figure 3-11. Some light fixtures in the main computer room appeared to lack safety wires

Figure 3-12. Halon tank in main computer room appeared to be secured to the wall

Standby emergency generator. A 800 kW Saturn Solar generator is installed at the facility, which we were told is of powering the entire GCF. It is supplied from a 2000-gallon tank outside the building (Figure 3-13), which is anchored to the slab, and capable of supplying the generator with 50 hr of fuel at 40 gal/hr. However, the fuel is delivered to the generator via a combined day tank and fuel pump (Figure 3-14). The fuel pump and day tank are not secured to the floor, which substantially increases the probability that in a strong earthquake they would slide and break the attached piping, itself secured to the wall, rendering the generator inoperative and possibly spilling diesel fuel (Figure 3-15). The generator requires batteries to power its starter motor. Although the battery rack is secured to the floor, the batteries lack spacers to prevent them from sliding within the rack (Figure 3-16), which increases the probability that movement of the batteries would break the conductors between individual cells, which would prevent the batteries from powering the starter motor, and prevent from the generator from starting. The generator is tested monthly, a desirable feature that mitigates the probability of operator failure or undetected equipment damage. However, the day tank's lack of anchorage and the battery rack's lack of spacers appear to be important weak links in the GCF's provision for on-site electric power in the event of the loss of off-site power.

Figure 3-13. GCF emergency generator (left) and its fuel tank (right)

Figure 3-14. GCF combined generator day tank and fuel pump, apparently not anchored to the slab

Figure 3-15. Piping attached to GCF generator day tank

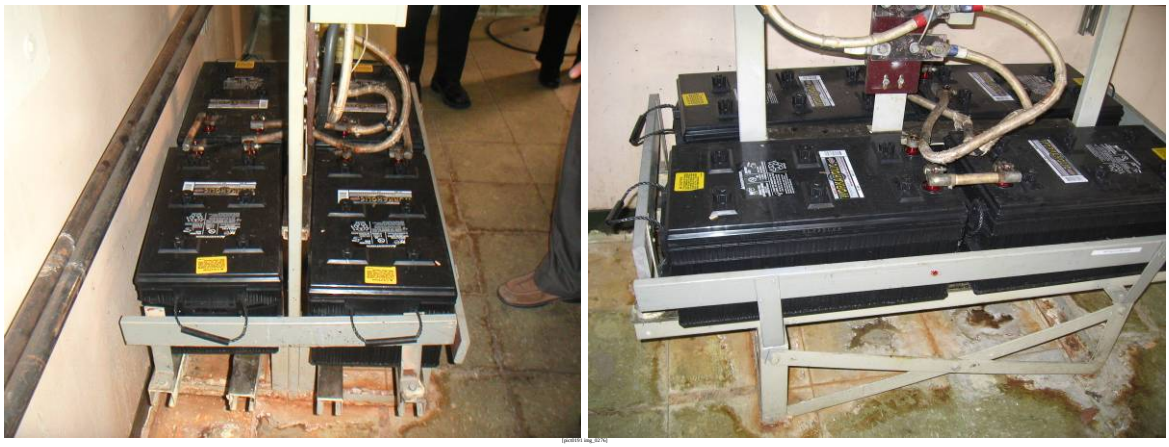

Figure 3-16. Battery rack for GCF generator starter motor was anchored but batteries lacked spacers

Other electrical equipment in electrical building. Uninterruptible power supply (UPS) equipment and other electrical equipment are housed in a small structure adjacent to the main building. The UPS battery racks are well anchored to the floor slab, have longitudinal diagonal braces and batteries are restrained from falling off the rack and supplied with spacers between batteries, all desirable features (Figure 3-17). Cabinets holding the transfer switches, as well as all other controls, breakers, and transformers whose anchorage was examined, appeared to be anchored to the floor slab (Figure 3-18). While overhead cable trays in at least one portion lacks lateral restraint, the run is braced in each direction by an abutting wall (Figure 3-19). There are some pendant lights in this room of a type that have collapsed in past earthquakes, but again, this mostly life-safety concern is mitigated by the fact that the room is not normally occupied (Figure 3-19). A radio tower on the top of this electrical building was not examined for seismic resistance (Figure 3-20).

Figure 3-17. GCF battery racks

Figure 3-18. Electrical cabinets appeared to be anchored to floor slab

Figure 3-19. Cable trays and lights in GCF electrical equipment room

Figure 3-20. Radio tower at GCF

3.2.2 Observations of equipment at data processing facility

This facility was visited on June 30, 2006. It serves data processing needs for the utility, including accounting, procurement, billing, and other crucial data processing functions. See Appendix B for the fault tree of this facility.

Telephone switching equipment. Telephone switching equipment in the telecommunications room appeared to be anchored at the base although without apparent bracing at the top (Figure 3-21). Cable trays lacked sway bracing, but were generally not in long straight runs, and did not appear to be particularly heavily loaded or capable of falling off their supports (Figure 3-22).

Figure 3-21. Anchorage of DPF telephone switching equipment

Figure 3-22. A DPF cable tray in a telephone switch room

Information Technology Operations Center (ITOC). Computer equipment in this room is mounted in long consoles, largely without straps or other means of preventing sliding or overturning of the system units or monitors (Figure 3-23). There appears to be no restraint of the consoles themselves to prevent sliding or overturning. Other furnishings in the room such as tall filing cabinets lack seismic restraint (Figure 3-24). Raised access floor in this room is on heavy 2-ft pedestals, but each pedestal appeared to be secured to the slab below by two

rather than four bolts, and does not appear to be braced, although nearby flooring in the adjacent ITOC equipment room is braced (Figure 3-25). We were informed that raised access flooring in this facility is in a multi-year process of seismic strengthening. Heavy equipment in the ITOC equipment room are provided with supplemental steel framing underneath the floor to prevent overturning, while lighter pieces of equipment are anchored to the slab beneath the raised access floor by vertical steel rods anchored into the slab below (Figure 3-26). Although the supplemental steel framing is intended to brace the equipment, not the floor, it might act to prevent excessive lateral motion of the floor. The vertical steel rods, if securely installed, would inhibit overturning of the equipment so long as the raised access floor did not displace or collapse. The lack of restraint for computers and consoles in the ITOC room, as well as the lack of bracing of the raised access floor, may represent weak links at this facility.

Figure 3-23. Consoles containing apparently unrestrained computer system units and monitors

Figure 3-24. Apparently unrestrained tall furnishings in DPF ITOC room

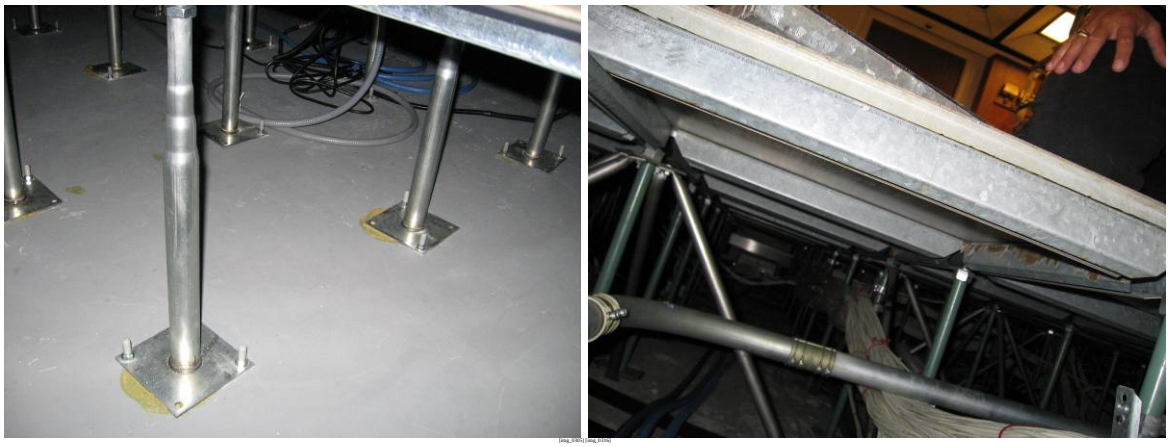

Figure 3-25. Raised access floor appears to be unbraced in ITOC room (left), although bracing is present in the ITOC equipment room (right).

Figure 3-26. Red members behind the floor pedestal are steel framing to support heavy equipment in ITOC equipment room (left); lighter equipment secured with vertical rods (right).

CPU rooms 1 and 2. Suspended ceilings examined here lack compression struts, and it was unclear whether light fixtures in the ceiling had secure, independent safety wires, which if absent would indicate a higher probability that light fixtures could fall, threatening life safety and endangering computer equipment. Computer equipment cabinets here are tethered at four corners to the slab below by various systems: slack steel cable with turnbuckles for tension adjustment, anchored to Unistrut members bolted to the slab below and secured by various means to the cabinet above (Figure 3-27); or by steel frames (Figure 3-28). Raised access floors in CPU room 2 are braced on one side of the room (Figure 3-29), although not on the other. Computer cabinets are all secured to the slab below by similar means as in CPU room 1, i.e., tethers or rigid frames, although it appears that the rigid frames in some cases rely on relatively long unbraced lengths of bolt (Figure 3-30).

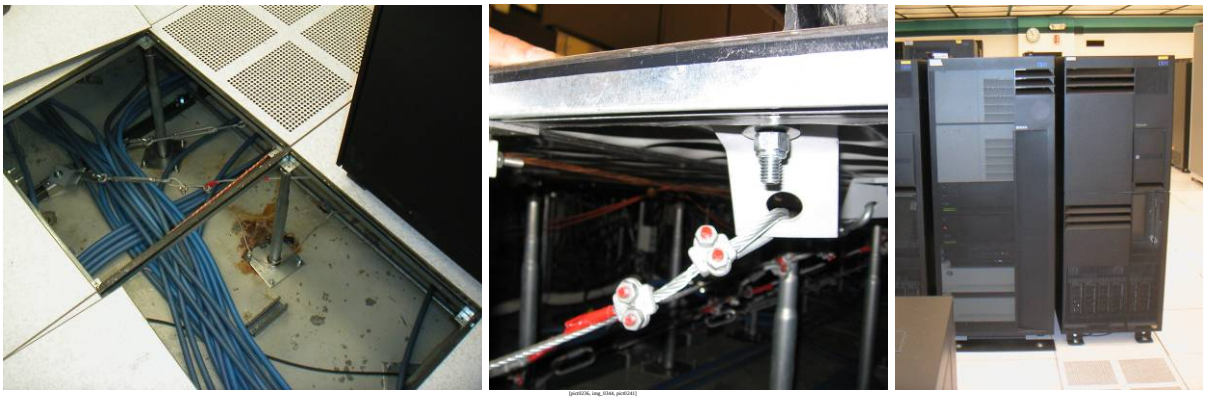

Figure 3-27. System of tethering equipment in CPU rooms 1 and 2

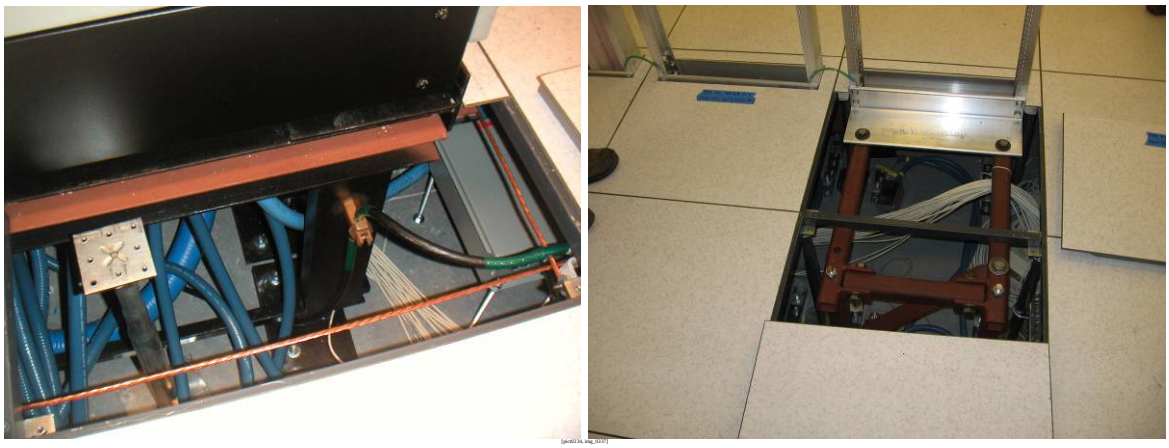

Figure 3-28. Steel frames used for some equipment in CPU rooms 1 and 2

Figure 3-29. Bracing on raised access flooring in part of CPU room 2

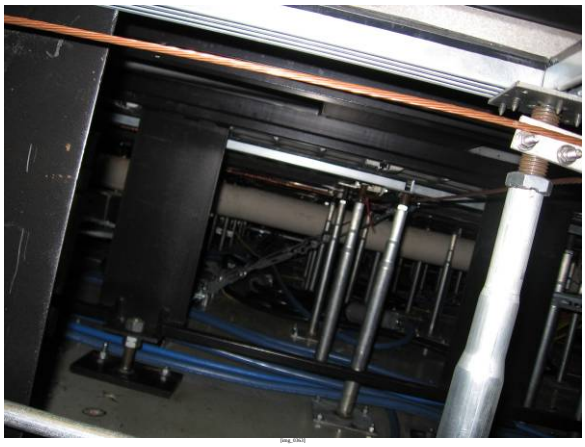

Figure 3-30. Seismic restraint of equipment in CPU room 2

Tape library. This room houses five cylindrical tape silos[3], as well as some smaller units that perform the same function (Figure 3-31). The tape silos are not secured to the floor, resting on leveling devices (Figure 3-32); in strong shaking, conduit attached to the tape silos from above (we were told these were for Halon supply; see the pipe above the cylindrical tape silo on Figure 3-31) would provide limited restraint, although these conduits are probably not intended to serve such a purpose. The raised access floors themselves lack diagonal bracing, and stand on slender pedestals secured to the slab with two shot anchors per pedestal (Figure 3-33). Halon tanks in this room are secured to the wall, and Halon pipes appeared to be at least partially braced (Figure 3-34). The lack of seismic restraint for the raised access floors and tape silos may represent a weak link at this facility.

Figure 3-31. Tape silos at DPF

[3] "Tape silo" is a misnomer; these devices handle hard disk drives.

Figure 3-32. Levelers under DPF tape silo

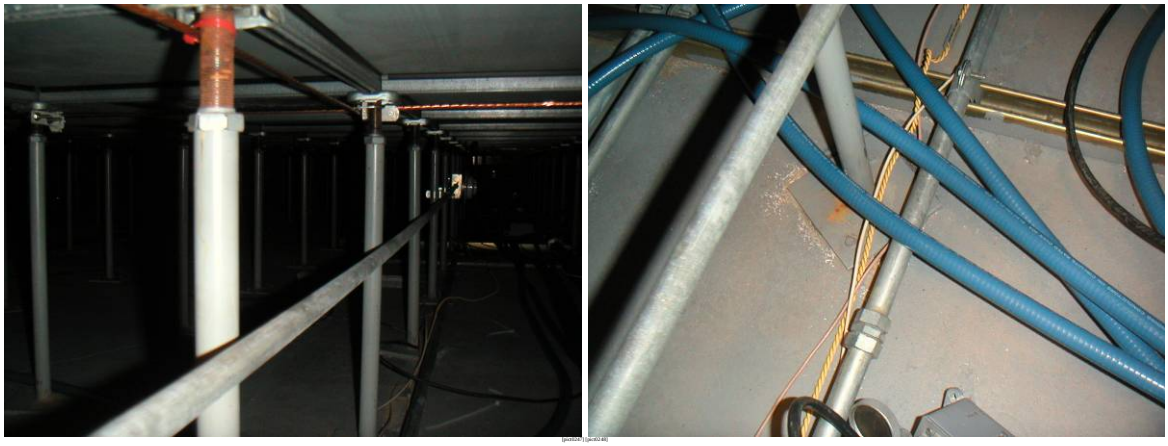

Figure 3-33. Raised access flooring in DPF tape silo room

Figure 3-34. Halon tanks and pipe in DPF tape silo room

Printer room. The printer room contains two large laser printers, which we were told are used to print reports and checks. The printers lack seismic restraint, which increases the probability that they would roll, slide, impact, or overturn in an earthquake, and become inoperative (Figure 3-35). Document insertion equipment (used for example to put checks in envelopes) were not present in this facility. This equipment stands on 2-ft raised access

flooring without diagonal bracing and limited anchorage to the slab below (two anchors per pedestal, as opposed to four; Figure 3-36). The lack of seismic restraint for the raised access floors and printers seems to represent another weak link at this facility.

Figure 3-35. DPF printer room equipment

Figure 3-36. Apparently unbraced raised access floor in DPF printer room

UPS equipment. The uninterruptible power supply (UPS) system, comprising two banks of two redundant systems (5 and 6, 7 and 8; Figure 3-37) appear to be anchored to the slab although not braced at the top (Figure 3-38); batteries appeared to be generally, but not always, secured on their racks, as shown in Figure 3-37. The cabinets that house the rectifiers, inverters, and other electric equipment needed to convert between outside power and UPS power appeared to be anchored to the slab (Figure 3-39). The electrical conduit between the batteries and these cabinets were supported on hangers that appeared to be braced to the wall (Figure 3-40).

Figure 3-37. DPF battery racks

Figure 3-38. Apparent anchorage at base (left) but not at top (right) of DPF battery-rack cabinets

Figure 3-39. UPS cabinets appeared to be anchored

Figure 3-40. UPS electrical conduit hangers in DPF apparently braced to the wall

Penthouse. It was unclear whether switchgear in the penthouse was anchored to the floor slab. The equipment cabinets were closed and building engineers accompanying us did not want to open them without the supervision of electricians. We assume these cabinets are well anchored, since virtually all other critical equipment in the facility is well anchored.

Breaker battery racks. We observed battery racks that building engineers suspected serve circuit breakers, as opposed to the main UPS (Figure 3-41). These racks have battery restraints and longitudinal braces but lack spacers between the batteries. Spacers (often Styrofoam sheets placed between batteries) are an attempt to prevent the batteries from shifting within their frame and breaking the conductors between adjacent batteries. The lack of spacers increases the probability that an earthquake would damage the conductors and prevent the battery rack from supplying power; this shortcoming may be a weak link at this facility.

Figure 3-41. Battery racks serving breakers

Second –floor mechanical room. Four redundant air handling units (AHUs) in this room represent an important exception to the seismic anchorage of equipment in this facility. They

are on vibration isolators without seismic snubbers (Figure 3-42). (Seismic snubbers are metal fixtures attached to the slab next to a vibration-isolated mechanical device to restrain the device's lateral movement; an example is shown in Figure 3-43). The AHUs are rigidly attached to piping that could be damaged by excessive displacement of the AHU in an earthquake (Figure 3-44), which is another issue that tends to raise the probability of operational failure. Note that two of the four AHUs are needed. Lack of seismic restraint for these AHUs is a potential weak link at this facility.

Figure 3-42. Air handling units in DPF on isolators apparently without seismic snubbers

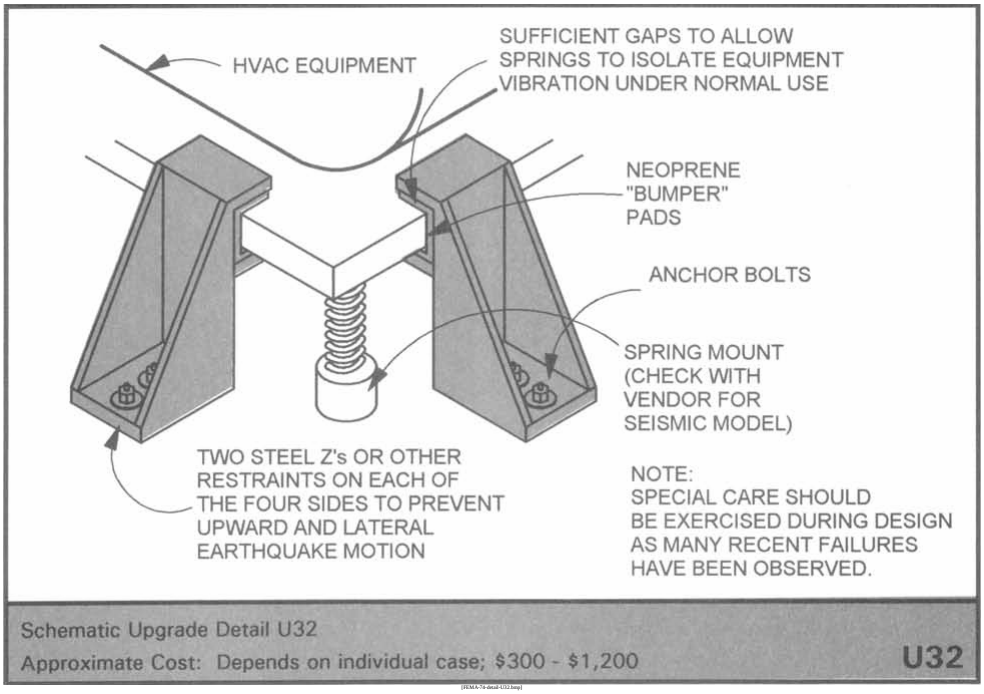

Figure 3-43. Schematic drawing of a seismic snubber, from FEMA (1994)

Figure 3-44. Piping rigidly attached to an air handling unit in DPF

Emergency generator. A diesel generator is located outside the building within an unroofed enclosure. The generator has a belly tank and a 10,000 main fuel tank capable of supplying the facility with electric power for 4 to 8 days. The generator, its fuel tank, and switchgear appear to be well anchored (Figure 3-45, Figure 3-46, Figure 3-47). Other nearby switchgear and transformers (Figure 3-48) appeared to be anchored to the slab.

Figure 3-45. DPF generator and its seismic restraint

Figure 3-46. DPF generator's fuel tank and its seismic restraint

Figure 3-47. Seismic restraint of DPF generator's switchgear

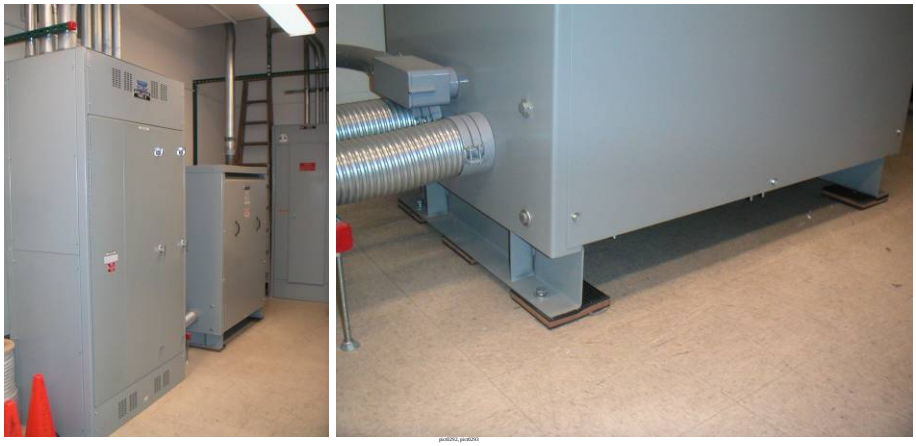

Figure 3-48. Anchorage of electrical equipment near DPF generator

Cooling towers at DPF appeared to be seismically anchored. DPF does not have an on-site water supply, and it is of concern that if the local water supply is interrupted in an earthquake, evaporation of the water in the cooling towers could render the air conditioning at this facility inoperative; this may be a weak link at this facility, which could be remediated by providing a backup water supply for the cooling towers.

Figure 3-49. Cooling towers at DPF and their anchorage. This facility lacks an on-site backup water supply in case the local water utility is damaged.

3.2.3 Observations of backup data facility equipment

This facility was visited on July 5, 2006. It serves as a backup site for the grid control and other data processing functions of both the grid control facility and data processing facility. See Appendix C for the fault tree of this facility.

Alternative grid control center. This room rests on 2-ft raised access flooring. It contains a number of 5-ft-high cubicles with desktop computers suspended beneath the desk and monitors secured to the tops of the desks. The cubicles are not mechanically restrained from sliding or overturning. If they were to slide or overturn, the electrical cords powering the computers could pull out of their sockets, rendering the affected AGCF computer equipment inoperative. Overhead is a suspended ceiling system with automatic sprinklers.

Second-floor utility space. The facility has three backup diesel generators (A, B, and C,), of which only one is needed to power the facility in the event of the loss of off-site power. The generators are all anchored to the floor slab with isolators that appear capable of providing substantial seismic resistance against uplift and lateral displacement (Figure 3-50). The day tanks and integral electric fuel pumps are secured to the slab as well (Figure 3-51). The batteries need to power the starter motors are not anchored to the slab, and lack battery spacers (Figure 3-52). They could slide off the plinth and possibly break their connection to the generator in the event of strong shaking, hindering the timely operation of backup generators. The lack of anchorage of starter-motor batteries and their lack of spacers may represent a weak link at the BDF facility.

Figure 3-50. Generators and their seismic isolators at BDF

Figure 3-51. BDF generator day tank and electric fuel pump, apparently anchored to the slab

Figure 3-52. Apparently unanchored starter-motor batteries for generators at BDF

Each generator has a 50-gal day tank and we were told burns approximately 110 gal/hr. A 20,000 gal fuel tank outside the building and anchored to its plinth (Figure 3-53) therefore provides up to 180 hr (8 days) of fuel. The fuel pump is inside the tank. We observed that the supply pipe that runs between the tank and the generators is secured along about 125 ft to a heavy concrete fence comprising seven panels with no connection between them (Figure 3-54). Differential displacement of the tops of these panels could conceivably rupture the supply pipe, both disrupting on-site power generation and causing a hazardous material spill. In that event the generator could possibly be supplied by manually filling 55-gallon drums (one was observed next to the tank) and hauling them up to the generator day tanks. An alternative approach would be to provide a shutoff valve on the pipe run from the tank and an alternative point for injecting fuel into the supply pipe from a tanker parked at the loading

dock, where the pipe enters the building (Figure 3-55). Failure of the diesel supply pipe along this fence may represent an important weak link at this facility.

Figure 3-53. Diesel fuel tank at BDF apparently anchored to its plinth

Figure 3-54. Fuel line secured to a long segment of concrete fence at BDF

Figure 3-55. Possible access point on the loading dock for alternative method of fueling diesel generators in case the fuel pipe breaks on the concrete fence

Near the generators is synchronizing gear, which ensures identical phase and voltage from the generators. The gear appears to be anchored to the slab. On the roof above, each generator has a radiator whose anchorage was not visible (Figure 3-56), muffler that appeared

to lack positive seismic restraint (Figure 3-57) and an enclosure containing air intake fan with equipment to cool the outside air before it is fed into the generator (Figure 3-58). The intake fan motor had vibration isolators with questionable resistance to lateral displacement. The lack of seismic restraint for the mufflers is presumably not a critical issue, as long as local authorities would allow the generators to be run despite damage to the mufflers. Damage to the air intake fans could conceivably prevent the generators from operating, so the light vibration isolators might be more of a reliability issue.

Figure 3-56. Radiator for BDF emergency generator

Figure 3-57. Muffler for BDF emergency generator

Figure 3-58. Enclosure with generator air supply fan and pre-cooling equipment. Isolator appears to have questionable seismic restraint

Cooling towers. There are two 1000-ton cooling towers at the facility with isolators that provide resistance to uplift and lateral displacement (Figure 3-59). There are two 30,000-gal water storage basins, each with three redundant pumps anchored to the floor (Figure 3-60). The cooling towers lose up to 500 gal/hr to evaporation, so if off-site water supply were lost, the basins could supply at least 60 hr of water for cooling. The basins can be refilled from trucks outside the facility.

Figure 3-59. BDF cooling towers and their anchorage

Figure 3-60. On-site water supply at BDF for use by cooling towers

Second-floor central plant. There are three redundant chillers, all with seismic anchorage (Figure 3-61). Compressors appeared to be seismically restrained (Figure 3-62). Hot air heating is electric, with two redundant and anchored 480V boilers (Figure 3-63), although air is heated solely for comfort and the boilers are therefore not critical equipment. Ventilation fans in this area were observed to be mounted on isolators that lack seismic restraint (Figure 3-64); building engineers informed us these are used for the warm-air system, and are therefore not critical to continued operations.

Figure 3-61. Chillers and their seismic anchorage in BDF

Figure 3-62. Compressors in BDF appeared to be seismically restrained

Figure 3-63. BDF electric boilers, apparently seismically anchored

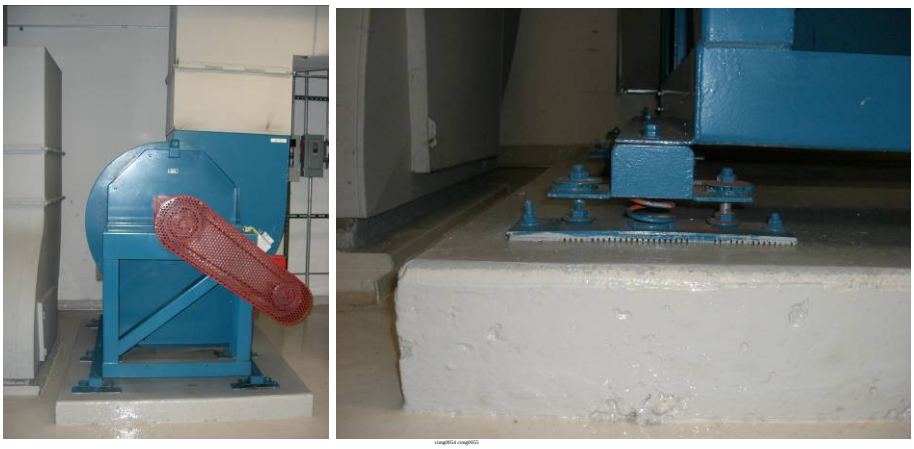

Figure 3-64. Fans in BDF for warm air, without apparent seismic restraint

Building control center. All building service equipment (mechanical, electrical, and plumbing) is controlled with direct digital control (DDC) from this second-floor room. Desks in this room are tethered to the slab beneath the raised access floor (Figure 3-65). The

monitors are not secured and at least some system units were in unrestrained cabinets (Figure 3-66). Suspended ceilings in this room appeared to have compression struts and diagonal splay wires, but not in the same places as would be required for modern construction (Figure 3-67). The ceiling is captured all the way around by walls, which tend to be more remediate this problem. However, ductwork and automatic sprinkler lines appeared largely to lack lateral restraint that would help to mitigate the potential for seismic damage (Figure 3-68). An equipment closet in this room contains approximately 20 non-redundant DDC panels fastened to the wall (Figure 3-69). Of some concern is the fact that this closet has automatic sprinklers rather than Halon. If the sprinkler pipe were damaged or water were otherwise discharged, it could conceivably damage all these panels. Although building engineers informed us that all the building service equipment can be run manually, the ceiling system in the control center, the sprinklers in this equipment closet, and the lack of seismic restraint for the computers in this room may represent important weak links at this facility.

Figure 3-65. Desks in BDF building control room appeared to be tethered to the slab

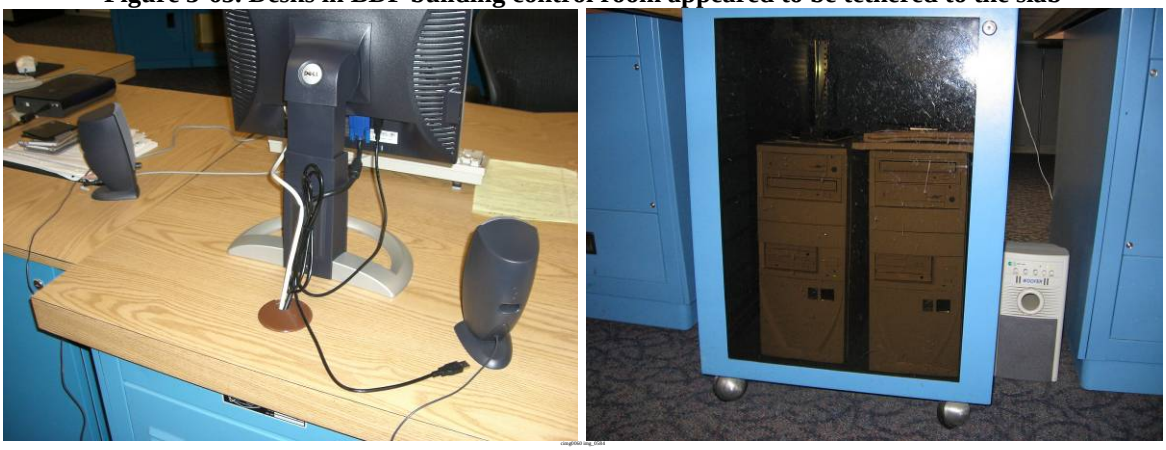

Figure 3-66. Some computer equipment in BDF building control room lack seismic restraint

Figure 3-67. Compression struts and diagonal splay wires in BDF building control room ceiling

Figure 3-68. Ductwork and automatic sprinkler lines in BDF building control room appeared largely to lack lateral restraint.

Figure 3-69. DDC console cabinet of the BDF building control room

Second-floor utility space. There are four redundant air handling units common to the primary spaces and an additional air handling unit for the printer room, all anchored to their plinths (Figure 3-70).

Figure 3-70. Air handling units in BDF appeared to be anchored to their plinths

UPS equipment. We observed four battery modules, with three cabinets per module, all apparently anchored to the slab (Figure 3-71). Batteries for the most part appeared to be secured to their racks (Figure 3-72). We did not observe whether the batteries have spacers, but as the image shows, conductors between batteries appeared to be flexible cable and thus less subject to fracture in case the batteries pound into each other inside the cabinet. We observed a (non-critical) battery charging rack without apparent anchorage (Figure 3-73). Two redundant systems of two uninterruptible power modules per system (rectifier, inverter, etc.) appeared to be anchored to the slab (Figure 3-74). Air conditioning equipment near the UPS systems appeared to be anchored to the slab (Figure 3-75).

Figure 3-71. UPS equipment apparently anchored to slab

Figure 3-72. UPS batteries at BDF

Figure 3-73. Non-critical battery charging rack at BDF without apparent anchorage to the floor

Figure 3-74. UPS equipment at BDF apparently anchored to the floor slab

Figure 3-75. Air conditioning equipment near BDF UPS cabinets, apparently anchored to the slab

Overhead electrical ducts were observed to be supported on braced hangers (Figure 3-76). However, the fire suppression system in the 2nd floor mechanical spaces is provided by automatic sprinklers (charged, wet lines), with long unbraced runs of pipe in excess of 20 ft. In several spaces these pipes were observed to be located near unbraced ductwork (Figure 3-77) that in an earthquake could interact with and possibly break the sprinkler pipe, which could lead to water spraying on UPS and other electrical equipment. This appeared to be a common situation throughout the building: braced electric conduit hangers and cable trays, but unbraced automatic sprinkler lines and air-conditioning ducts. The potential for earthquake sprinkler leakage caused by damage to sprinkler pipe may be an important weak link at this facility, especially in areas with electrical equipment. Damage to air ducts aggravated by lack of bracing could also pose a serious problem for air conditioning in computer spaces.

Figure 3-76. Overhead electrical ducts and hanger braces at BDF

Figure 3-77. Long unbraced sprinkler lines and places where they could interact with ductwork

Switchgear room. We observed two long panels of switchgear, all apparently anchored to the concrete plinth (Figure 3-78), and two racks of batteries to power these, also anchored to the slab and having longitudinal bracing and no need for battery spacers (Figure 3-79).

Figure 3-78. BDF switchgear and its anchorage

Figure 3-79. Battery racks, apparently well anchored, with restrained batteries and longitudinal braces

Telecommunications room. This room houses telecommunications equipment belonging to the facility's owner (as opposed to the local telephone service provider). The

telecommunication equipment appeared to be secured to the floor slab (Figure 3-80). Overhead cable trays appeared to be braced to the wall (Figure 3-81). The room is protected by automatic sprinklers as well as a Halon system, although building engineers informed us that the Halon system should suppress any fire and prevent the fusible links on the automatic sprinklers from melting and causing water to discharge, although as elsewhere long unbraced runs of pipe (Figure 3-82) raise increase the probability of earthquake sprinkler leakage. The Halon tank appeared to be secured to the wall (Figure 3-83). The telecommunications switching equipment has its own battery rack, anchored to the slab, with longitudinal braces and battery restraints and spacers, all desirable features for seismic resistance (Figure 3-84).

Figure 3-80. Telecommunications equipment at BDF and its anchorage

Figure 3-81. Overhead cable trays in BDF telecommunications room appeared to be braced to the wall

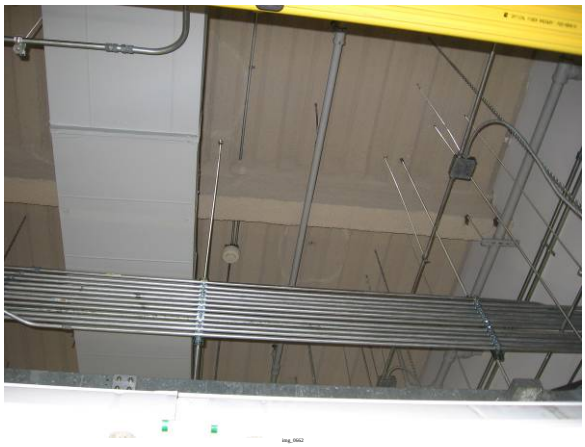

Figure 3-82. Long unbraced runs of sprinkler pipe in BDF telecommunications room

Figure 3-83. Halon tank in BDF telecommunications room, apparently strapped to the wall

Figure 3-84. Battery rack for telecommunications equipment

Transformer vault. Outside the facility were four transformers that we were told step down power from the 16000 kV train from the nearby substation to 480V. There are two transformers per each of two trains. The vaults were not directly accessible at the time of our

visit, but equipment near a vantage point appeared to be restrained from sliding (Figure 3-85).

Figure 3-85. Transformer vault at BDF, and apparent seismic restraint of its equipment

CPU room. This large room stands on 2-ft raised access flooring. The pedestals within sight of our vantage point lack diagonal bracing, and were anchored at four corners with epoxy anchors (Figure 3-86). Equipment in this room is restrained by various systems, for example, bolted to IBM anti-tip plates in front and an innovative angle bracket to a rear wheel axle. These are then secured to a steel cable tether, with turnbuckle to adjust for tautness, itself secured to a Unistrut member secured to the slab below (Figure 3-87). We were told that all equipment in this room weighing in excess of 400 lb is tethered in this fashion. Lighter equipment is secured against overturning by threaded rods secured to the levelers and anchored to the slab below, as at DPF. It is conceivable that the tethers on the larger equipment could act to restrain the raised access floor in the absence of diagonal braces, but because the tethers are not taut, and are not designed to restrain the floor, the raised access flooring is more likely to collapse than if diagonal braces were present. Unbraced raised access flooring could be an important weak link at this facility.

Figure 3-86. Raised access floors at BDF CPU room, apparently lacking braces

Figure 3-87. System of tethering computer equipment to slab below raised access floor in BDF CPU room

The ceiling in the CPU room was observed to have compression struts and splay wires, and the light fixtures near our point of inspection had safety wires (Figure 3-88). At least some ductwork within view of this point also appeared to be braced, but long unbraced runs of sprinkler pipe were also visible (Figure 3-89).

Figure 3-88. Ceiling in the BDF CPU room with compression struts, splay wires, and safety wires on light fixtures

Figure 3-89. Ductwork and sprinkler pipe at BDF CPU room

Helpdesk room. This room stands on 2-ft raised access flooring. Cubicles appeared to lack seismic restraint. Each cubicle had one or more desktop computers on the floor or the desk, with one or more CRT or LCD monitors, none of which appeared to be secured (Figure 3-90). Sliding or overturning of cubicles and computer equipment, and possible damage or pullout of electrical connections, are all more likely because of the lack of seismic restraint. Vulnerability of the helpdesk facility may represent an important weak link.

Figure 3-90. BDF helpdesk center and apparently unsecured computer equipment

Tape silo room. Similar to the data processing facility, four cylindrical tape silos in the backup data facility rest on 12-in raised access floor. While some controllers appeared to be seismically restrained (Figure 3-91), the cylindrical silos themselves are not, and are therefore more likely to slide, pound against each other, and possibly become inoperative in an earthquake (Figure 3-92). This may be an important weak link at this facility.

Figure 3-91. Seismic restraint of controllers in BDF tape silo room

Figure 3-92. Apparently unrestrained BDF tape silos

Print room and document handling equipment. The print and distribution area stands on 1-ft raised access flooring. Equipment in this room is tethered to the floor panels (Figure 3-93); we do not know whether the bolts connected to the floor panel are themselves restrained to the slab below using a tethering system such as described above. Document insertion equipment in the warehouse (the alternate mailing operation center; no raised access floor) appears to be secured to the slab (Figure 3-94). Long unbraced lengths of sprinkler pipes and or minimally braced copper air lines were observed suspended from the deck above (Figure 3-95). Compressed air tanks and compressors (compressed air being needed by the document handling equipment) appeared to be anchored to the floor slab (Figure 3-96).

Figure 3-93. Document handling equipment in BDF restrained to raised access floor

Figure 3-94. Restraint of document handling equipment in BDF

Figure 3-95. Long unbraced lengths of sprinkler pipe and minimally braced copper air pipe in BDF

Figure 3-96. Anchorage of air tanks and compressor assemblies in BDF

3.3 OTHER DATA COLLECTION

3.3.1 Off-site utility failure

We reviewed available literature to estimate the failure probability of off-site utility services, most notably domestic water (most relevant to the continued operation of cooling towers) and offsite electric power.

3.3.2 Fragility of unusual equipment

Approximately cylindrical tape silos are present at DPF and BDF. These devices are approximately 5 ft in diameter, 6 ft tall, and hold between 500 and 1,000 data cartridges. The specimens observed rest on metal levelers on smooth raised access flooring, without positive seismic restraint, although Halon conduits enter the device from above, and might provide a stiff but perhaps brittle point of restraint. No seismic fragility information was readily available about these devices. They do not appear in Johnson et al.'s (1999) manual, and no evidence of them appears in an important database of past nonstructural damage (Kao et al.

1999). The devices may be relatively rare (especially in comparison with switchgear, cooling towers, etc.) or have not yet been observed to experience damage in earthquakes (also reasonable in that they fairly new). We therefore formulated fragility functions for them.

3.4 STRUCTURAL VULNERABILITY ANALYSIS

3.4.1 Methodology

Combination of two national standards. There is no single standard methodology to relate the earthquake shaking intensity a building experiences to the likelihood that a building safety inspector will subsequently post the building unsafe to enter or occupy. To develop such a relationship, one can perform a two-step procedure: use a standard engineering methodology to relate shaking intensity to the degree of a building's structural damage (here, either FEMA-356 or HAZUS), and then apply another standard (ATC-20) to relate damage to the outcome of the safety inspection. Such a procedure is easier to understand by considering the second step first.

ATC-20 Safety Evaluation. ATC-20 (Applied Technology Council 1996) represents the standard methodology for post-earthquake safety evaluation of buildings, in which inspectors examining a building after an earthquake estimate the building's seismic safety and determine whether the building is safe to enter or occupy. The portion of ATC-20 that is relevant here is the conditions under which the procedure calls for a building safety inspector to post a building as unsafe (i.e., to red tag the building). The document requires that the inspector investigate the building for each of several conditions:

- Collapse, partial collapse, or building off foundation
- Building or story leaning
- Racking damage to walls, other structural damage
- Chimney, parapet, or other falling hazard
- Ground slope movement or cracking

The inspector can judge each condition to be in any of three degrees: minor or none; moderate, or severe, with the judgment of what constitutes each degree left to the inspector. The inspector is instructed that "Severe conditions *endangering the overall building* are grounds for an Unsafe posting. Localized Severe and overall Moderate conditions may allow

a Restricted Use [yellow tag] posting." [Emphasis added.] Otherwise the building is posted "inspected," (green tag), meaning that inspector has placed no restriction on the building's use or occupancy.

Note that ATC-20 begins with an observation of damage, and does not include structural analysis or consider the shaking intensity that the building experienced, those issues being largely irrelevant once the damage has occurred. Hence the need for an analytical methodology to estimate the damage before the earthquake has occurred. Two such methodologies exist that can be considered national standards: FEMA 356 (published by the American Society of Civil Engineers in 2000), and HAZUS (Federal Emergency Management Agency and the National Institute of Building Sciences 2003). The former applies to individual buildings and includes a detailed structural analysis and assignment of a "seismic performance level." The latter is more generic, intended for estimating the seismic performance of large numbers of buildings where detailed structural analysis is impractical.

We have used both methodologies: FEMA 356 for the data processing facility and backup data facility (the facilities for which we were able to perform detailed structural analyses), and HAZUS for the grid control facility.

FEMA 356 for Building-Specific Structural Damage Analysis. The FEMA 356 methodology was developed for assessing seismic rehabilitation measures for existing buildings. The relevant portion of the methodology is its procedures for creating a mathematical model of an existing building or proposed retrofit, performing a structural analysis, and estimating the structural damage to the building. A product of the analysis is the estimated damage to each structural component, assessed in terms of whether the component meets acceptance criteria for three seismic performance levels: immediate occupancy, life safety, and collapse prevention.

The FEMA 356 and ATC-20 criteria are defined somewhat qualitatively, and do not use the same terminology, so there is no definitive way to mesh FEMA 356 and ATC-20, no authoritative relationship between FEMA-356 acceptance criteria and ATC-20 tag color. However, a reasonable interpretation of FEMA 356 and ATC-20 is that a building with "Localized Severe and overall Moderate" damage might refer to a building with up to 5% or so structural elements failing the FEMA 356 life-safety acceptance criteria, and that "Severe conditions endangering the overall building" could reasonably refer to a building with between 5% and 15% of its structural elements failing the FEMA 356 life-safety acceptance

criteria. We therefore judgmentally the relationship between the fraction of structural components failing the FEMA-356 life-safety criteria and the probability of being red-tagged as shown in Figure 3-97.

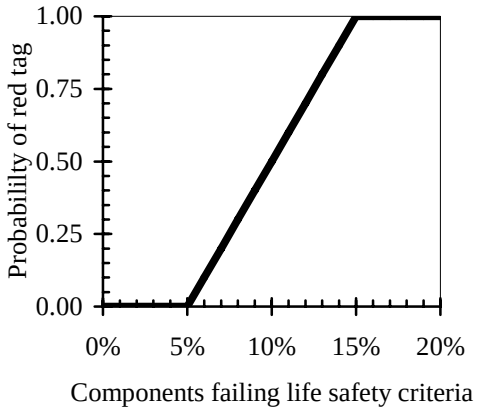

Figure 3-97. Relating FEMA-356 structural-damage assessment and ATC-20 safety evaluation

HAZUS for Generic Structural Damage Analysis. The HAZUS methodology represents a national standard in the estimation of the future seismic performance of generic buildings. It is applied here to the grid control facility, which is treated as a generic lowrise reinforced concrete shearwall building designed to moderate seismic code requirements. The portion of the HAZUS methodology that is relevant here for estimating the probability of red-tagging is the procedure for estimating the damage state of the building's structural components. By "damage state" is meant one of six qualitative degrees of damage: none, slight, moderate, extensive, complete, and collapse. A building in the "complete" or "collapse" damage state can be certainly associated with a red tag.

Would a building in HAZUS' *extensive* structural damage state be red tagged? The HAZUS Technical Manual describes extensive damage in several cases as potentially involving partial collapse, where "Some of the frame elements have reached their ultimate capacity" and other locally severe conditions. A reasonable interpretation of partial collapse is a "Localized severe condition," which ATC-20 suggests as grounds for a Restricted Use (yellow tag) posting, rather than red. We therefore judge that red tagging corresponds to the complete or collapse damage states, but not to the extensive damage state.

3.4.2 Assessment of data processing facility structural system

The data processing facility is a single story building comprised of two parts – a computer area on the west and an office area on the east. Both areas have small penthouses

above the roof of the single story. The original structural drawings of the building are dated 1973 and the building was probably designed according to the 1970 Uniform Building Code (UBC) provisions. The 56,320 square-foot computer area is a box structure (256 feet by 220 feet) fully enclosed by double-layered reinforced concrete shear walls on the perimeter. The two layers of shear walls are 9-1/4-inch and 10-inch thick, and are separated by a 4-3/4-inch gap filled with Styrofoam. Reinforcement is provided either in the form of a single layer #4 horizontal and vertical rebar mesh spaced at 8 inches or two layers with 16 inch spacing. There are also some 12-inch thick interior shear walls. The foundation underneath the computer area consists of drilled concrete-filled steel piles grouped together in pile groups with 40-inch to 58-inch deep pile caps. The pile caps are tied together by a network of tie beams. Lap splices with splice lengths of 36 times the bar diameter are provided for the wall rebar above the pile cap. The dowels are embedded a distance of 24 times the bar diameter into the pile cap. This indicates that the shear walls are quite well connected to the foundation in the computer area. The reinforced concrete slab-on-grade is 6 inches thick. The roof slab is a reinforced concrete waffle slab with 30-inch by 30-inch pans typically, with a joist depth of 2 ft 8 inches. The minimum thickness of the slab is 12 inches. It is reinforced with #4 rebar at 12 inches on center each way at the bottom, and 6x6 welded wire mesh at the top. The penthouse, with an area of 12,312 square feet, is enclosed by 10 inch thick shear walls that rest on beams at the roof slab of the main structure. The wall in combination with the floor beam acts as a deep beam spanning between gravity columns. Interior gravity columns supporting the roof are made of reinforced concrete and are square or rectangular in shape with dimensions varying between 24 inches by 24 inches and 40 inches by 40 inches. The lateral force-resisting system of the computer area, comprising of the long perimeter double-layered shear walls and a very stiff rigid diaphragm in the form of the waffle slab, is so robust that there is little likelihood, if any, of the computer area of the building being red-tagged following an earthquake. Hence, no detailed analysis is performed of the computer area of the structure.

The lateral force-resisting system of the office area, however, is unlike that of the computer area and needs close evaluation. The office area is essentially a steel structure with concrete encasement for a few key members. It is a very light structure. The roof slab of the single-story structure is composed of vermiculite concrete on metal deck with a weight of just 9 pounds per square foot The weight of the supporting steel framework is about 8 pounds per square foot for a total roof weight of just 17 pounds per square foot. The office area is

bordered on the west by the double-layered reinforced concrete shear wall of the computer area structure. The lateral force-resisting system of the office area consists of perimeter steel moment-resisting frames on the remaining three faces of the building and the computer area shear wall on the west face. The moment-frame columns consist of steel members (size W14x228) typically with box sections at the north-east and south-east corners. The moment-frame beams consist of steel W30x99 sections embedded in 7-foot-2-inch deep by 2-foot-0-inch wide reinforced concrete spandrel beams. These spandrels are far stiffer and stronger than the columns, and as a result any earthquake damage to the frames is likely to take the form of plastic hinges at the top (below the bottom of the spandrel beam) and bottom of columns. Plastic-hinge formation in columns prior to the formation of hinges in the beams is considered a nonductile failure mechanism and is not desirable for earthquake resistance. The yielding in the columns could significantly reduce their gravity load-carrying capacity risking the ability of the structure to remain stable under its own self-weight. In 1973 (around the time the building was constructed), the concept of ductility was not well-understood. Present-day standard practice and code requirement of strong column-weak beam had not yet been adopted. The presence of the spandrels, presumably for architectural purposes, hinders the flexural rotation at the beam-column joint needed for the ductile response of a moment-frame system. Even in the absence of the spandrels, the moment-frame system used to resist lateral forces in the office area would have required ultrasonic testing of the welds connecting the beams to the columns and visual inspection of other features that caused brittle cracking in many steel moment-frames during the 1994 Northridge Earthquake, especially in frames with deep beams of the kind used in this building. The presence of the spandrels, while possibly protecting the beam-column connection from being loaded severely, would transfer the locations of the plastic hinges to the columns. This is an equally undesirable failure mechanism if not more than the brittle beam-column weld fracture mechanism.

While the spandrels are well reinforced with four #11 top and bottom bars and #5 bars at 10 inches side-face bars and two-piece overlapping #5 stirrups (with 90 degree bends) spaced at 10 inches, concrete spalling is still possible under a strong earthquake as a result of insufficient confinement. It should be mentioned, however, that despite these weaknesses, the magnitude 5.9 Whittier Narrows earthquake of 1987 that occurred at a distance of less than 1 mile from the building reportedly caused no visible damage to the structure, according to a past study of the building provided by the utility. The roof slab is anchored into the

concrete spandrels on three of the building faces and into the computer area shear wall on the west face. The connection consists of a ¼-inch-thick bent steel plate with two ¾-inch-diameter anchor bolts spaced at 24 inches on center typically and embedded 6 inches into the concrete. The horizontal leg of the bent plate is welded to the metal deck. The roof slab in the computer area is at a different elevation than the roof slab in the office area. As a result, while there is some chance of pounding occurring in the east-west direction if large roof displacements occur, it is unlikely that there would be any load-sharing between the office area structure and the computer area structure in the east-west direction. In the north-south direction however, the shear wall at the interface will provide lateral resistance to the motion of the office area structure. No shear studs have been provided in the beams supporting the roof slab. The metal deck is welded to the steel beams. A few struts and a couple of horizontal steel braces are provided to improve the performance of the roof diaphragm, especially at the west end where it is attached to the computer area shear wall. The steel columns are founded on spread footings. At the west end of the single-story office structure, a penthouse (area of 2,223 square feet) rises above the roof. It is supported by shear walls on all four sides. The penthouse floor slab is 12 inch thick and is made of reinforced concrete with #5 rebar spaced at 10 inches in the short direction and at 18 inches in the long direction top and bottom. The penthouse roof is similar to the main roof, consisting of vermiculite concrete on metal deck supported by steel framework.

2. Other known details about the structure and its foundations

The following are some key details of the structural design of the data processing facility:

(a) Possibly designed per Uniform Building Code, UBC 1970.

(b) Design loads: roof live load unknown;

(c) Built areas: computer area of 56,320 square feet at the 1st floor and roof, with a 12,312 square foot penthouse; office area of 22,400 square feet at the 1st floor and roof, with a 2,223 square-foot penthouse; overall dimensions of the computer area: 256 feet (north-south) by 220 feet (east-west); overall dimensions of the office area: 160 feet (north-south) by 152 feet (east-west); nominal story height is 17 feet 8 inches in the computer area and 14 feet 10 inches in the office area.

(d) A 1972 foundation investigation report provided by the utility indicates that the site is underlain by fill soils, one to six feet in thickness, consisting of silt, clay, sand, silty sand, and

clayey sand. The upper natural soils were only moderately firm at the moisture content determined from exploration borings. The soils were found to be generally firm below a depth of ten feet, but contained some softer layers. Water was measured in the borings at depths of 20 to 24 feet below the existing grade. The report recommended the use of driven friction piling to provide support for the building with minimum settlement. Compaction was recommended for the soil supporting the slab on grade.

(e) Structural drawings provide three options for drilled piers: concrete-filled step-tapered steel shell pile with diameter varying from 9-3/8-inch at the bottom to 14-1/8-inch at the top, concrete-filled steel pipe with a diameter of 12 inches, and precast prestressed concrete pile with a diameter of 12 inches – each with a downward load-carrying capacity of 160,000 pounds. The foundation investigation report suggested a lateral load-carrying capacity of 12,000 pounds for 16-inch diameter piles.

(f) Rebar #6 and above: ASTM A432 Grade 60; #3 through #5 bars: ASTM A615 Grade 40; wire mesh: ASTM A185.

(g) Reinforced concrete with 28-day compressive cylinder strength of 3,000 pounds per square inch.

(h) Light-weight vermiculite concrete 28-day compressive cylinder strength of 150 pounds per square inch.

(i) Structural steel: ASTM A36; Typical bolts: ¾-inch diameter ASTM A-307 typical; welding: E60 or E70 series electrodes.

(j) There are no seismic joints in the building.

For the data processing facility, we estimate that the peak ground acceleration corresponding to a 1% probability of this facility being red-tagged is in excess of 3g; none of the scenario earthquakes examined here is estimated to produce anywhere near this level of shaking. The probability of a red tag if the data processing facility experienced 0.9g of shaking (the median PGA in the most severe scenario event considered here) is estimated to be less than 1 in 1 million, using the methodology described above.

3.4.3 Qualitative assessment of grid control facility structural system

Following is a general description of the structural components of the facility and observations of potential deficiencies.

(a) The original structural drawings (3 in number: 547550-547552) are dated 1957.

(b) The structure consists of a foundation/first floor, a mezzanine second floor (covering only part of the overall plan area), and a roof. It is a box-like structure with 12-inch-thick reinforced concrete shear walls on all four faces on the perimeter.

(c) The first floor measures 63 feet (N-S direction) by 74 feet (E-W direction), or 4,662 total sq. ft. The second floor (on the western half of the structure) measures 63 feet (N-S direction) by 31 ft 7 inches (E-W direction), or 1,990 total sq. ft. There are two small areas on the north-east and south-east corners that house mechanical equipment (about 16 ft 8 inches by 14 ft 9 inches and 11 ft 5 inches by 11 ft 8 inches, respectively). These areas are bounded on two sides by the perimeter shear walls and are enclosed by 8-inch-thick walls elsewhere. The roof area is the same as first floor area.

(d) The second floor slab braces the west shear wall along its full length, but only about half of the north and south shear walls. The north-east and south-east corners of the perimeter shear walls are also braced by the mechanical slabs.

(e) The floor-to-floor heights are 10 ft 11 inches at the first story and 12 ft 9 inches at the second story. Thus the perimeter shear walls are unbraced a total height of about 23 ft 8 inches at some locations.

(f) The slab-on-grade is 6 inches thick with one layer of #3 rebar at 18 inch centers each way.

(g) The second-floor slab consists of 3-1/2-inch-thick reinforced concrete slab supported by 4 inch wide joist-like bridging (1 ft, 5-1/2-inch deep). The bridging has one #4 bar top and bottom. The bridging runs east-west (slab spans one-way in the north-south direction). Slab reinforcement consists of #3 bars at 15 inches on center (one layer) in the span direction and one #3 bar each side of the joist in the orthogonal direction. The east ends of the joists connect to a steel girder (27 WF 94) that is encased in concrete. The steel girder is supported by three steel gravity columns (10 WF 89), one in the interior encased in concrete and two embedded within the north and south shear walls.

(h) The roof slab consists of 6-inch-thick concrete spanning one-way in the north-south direction and supported by concrete beams that connect to the perimeter east and west shear walls at one end and an interior steel plate girder at the other end. The steel plate girder is encased in concrete and connects to the three steel columns. Slab reinforcement

consists of #4 bars at 7-inch centers bottom at mid-span in the span direction and #4 bars at 11-inch centers top at the support locations. In addition, the bottom bars are bent upward at the supports to increase the support top reinforcement.

(i) Shear-wall reinforcement consists of #4 bars at 13-inch centers each way, each face, typically. Dowels have been shown for shear wall connection with foundation. Slab dowels (#3 bars at 18-inch centers) are shown between the shear walls and the slab-on-grade. Slab top reinforcement at the second floor and roof has been shown hooked into the shear walls. No boundary elements have been provided in the shear walls and probably not needed in a squat wall situation such as this. General notes specify minimum splices and laps in reinforcing to be as follows: (i) Top bars, beams, and girders: 36 x bar diameter. (ii) Column verticals: 30 x bar diameter. (iii) Others: 25 x bar diameter. All of these seem adequate.

(j) Concrete strength used: 2000 psi at 28-days.

(k) Steel used: ASTM A-15, intermediate grade steel.

(l) Foundation: spread footings with bearing value of 2500 psf.

The following potential deficiencies were identified:

(a) Reinforcement in shear walls is less than minimum requirements specified in current code (ACI 318-2005). The behavior of the shear walls is potentially not ductile enough.

(b) One-foot-thick shear walls are unsupported for a height of 23 ft 9 inches for some length of the wall (about 30 feet long). Out-of-plane stability of walls should be evaluated.

3.4.4 Qualitative assessment of backup data facility structural system

The lateral force-resisting system is a mixed system with braced frames in the second story transferring the seismic forces to precast concrete wall panels at the first story in the north-south direction, and to precast wall panels as well as a couple of cast-in-place shear wall segments at the first story in the east-west direction. The braces consist of 10-inch tube steel sections and can be expected to perform reasonably well with regard to resisting buckling.

Both the braced frames and the precast wall panels are well distributed in both directions at the second story and first story, respectively. There is a good amount of redundancy in the lateral force-resisting elements with nine braced frame bays in the short direction (east-west),

and eight braced frame bays in the long direction (north-south) at the second story. Likewise, the structure is largely wrapped up by the precast wall panels which double up as the façade of the structure at the first story. The lateral force-resisting system is stiffer, and possibly more ductile and stronger as well in the east-west direction (short direction of the building) as a result of the presence of the 12-inch-thick cast-in-place shear walls in the interior of the building, in addition to the precast wall panels on the perimeter.

The performance of buildings with concrete tilt-up shear wall panels in the 1971 San Fernando Valley earthquake and the 1994 Northridge earthquake has been found to be poor with many instances of total building collapses as a result of the failure of the panel-to-floor slab connections that are typically embedded plates or angles anchored into the panels through studs and welded to the end angle supporting the concrete slab on metal deck. These anchors in concrete exhibit low ductility and the repeat cyclic loading from an earthquake can quite easily pry the anchors loose leading to a brittle failure of the connection, and subsequent collapse of the shear wall panels. The following excerpt is taken from a March 1994 EQE summary report following the Northridge earthquake:

> **Several new business parks throughout the San Fernando Valley were severely damaged. One business park had extensive structural damage to all of its roughly one dozen buildings. The two-story structures were steel framed and were enclosed with two-story-high, reinforced concrete, precast tilt-up panels. The first-floor sections of the tilt-up panels failed, and finished elements of the building interiors were extensively damaged [see Figure 3-98 and Figure 3-99]. Prior to the earthquake, the buildings housed computer centers and other service operations. One week after the earthquake, most of the buildings had been completely evacuated.**

Figure 3-98. Damage to tilt-up construction during the Northridge earthquake – this section of tilt-up wall pulled away from the rest of this high-technology company's corporate computer center. There was severe damage to the interior as a result. Source: EQE International (1994).

Figure 3-99. Damage to tilt-up construction during the Northridge earthquake – The wall of this office tilt-up nearly fell away during the earthquake. It has been braced, and a temporary tie to the adjoining wall has been installed until more permanent repair can be made. The interior of the second floor was almost completely destroyed (as shown in the photograph on the left). Source: EQE International (1994).

There are many such instances of the poor performance of precast tilt-up construction during earthquakes and for a critical facility, a more robust and ductile structural system should be in place.

Finally, it is to be noted that the concrete slabs at the second floor and the roof are sturdy enough to act as rigid diaphragms and can be expected to transfer the seismic inertial forces reliably to the vertical components of the lateral force-resisting system and retrofit schemes can certainly take advantage of this fact.

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4 RISK ANALYSIS

4.1 RISK ANALYSIS METHODOLOGY

4.1.1 Purpose of the risk analysis

The purpose of the risk analysis is to estimate two probabilities: (1) that operational failure occurs to both GCF and BDF in a single earthquake at least once during the next 5 years, and (2) that operational failure occurs to both DPF and BDF in a single earthquake at least once during the next five years. The risk analysis uses as its input the products of the seismic hazard analysis and the vulnerability analysis.

4.1.2 Overview of the risk analysis mathematics

To perform the risk analysis, we combine the results of the seismic hazard analysis and the vulnerability analysis as follows. For the benefit of the nontechnical reader we summarize the methodology as follows.

From the seismic hazard analysis, we have for each earthquake scenario an estimate of the probability distribution of shaking at each site. From the vulnerability analysis, we have estimated the probability of operational failure, given any particular level of shaking intensity. We multiply the failure probability given a particular level of shaking intensity, times the probability of the site experiencing exactly that level of shaking intensity in that earthquake, and sum over all possible shaking intensities in that earthquake. The result is an estimate of the failure probability for each facility, given the earthquake scenario. If we denote by P_A the probability that facility A fails in the earthquake, P_B denotes the failure probability of facility B in the same earthquake, and P_{AB} denotes the probability that both facilities A and B fail in the same earthquake, then we can estimate P_{AB} as

$$P_{AB} = P_A \text{ x } P_B \tag{4}$$

This equation assumes that the failure of the two facilities is independent, that is, given the shaking at each site, the failure of one facility has no effect on the failure probability of the other, a reasonable assumption where the facilities are not identical and are not construction by the same contractor at the same time. For each earthquake scenario we also have estimated the mean annual rate at which such a scenario occurs; let that rate be denoted by R_i, where i is an index to denote the scenario (a counting number 1, 2, … N), and where N

denotes the number of earthquake scenarios considered. We multiply the failure probability given the scenario earthquake, P_{AB}, times the rate at which the scenario earthquake occurs, R_i, and sum over all scenario earthquakes. Letting R denote the total mean annual rate at which both facilities A and B fail in a single earthquake, and letting $P_{AB,i}$ denote the failure probability for scenario i, we can estimate

$$R = R_1 \text{ x } P_{AB,1} + R_2 \text{ x } P_{AB,2} + \ldots R_N \text{ x } P_{AB,N} \tag{5}$$

If we denote by P_T the probability that both facilities 1 and 2 will fail at least once in a single earthquake during a period of T years (we use T = 5 years), then P_T can be estimated as

$$P_{T=5} = 1 - e^{-5R} \tag{6}$$

where e denotes Euler's number, approximately 2.718. For example, if R = 0.01 (i.e., failure will occur on average 0.01 times per year, or on average once every 100 years), then in 5 years the probability that facilities 1 and 2 will both fail in a single earthquake is given by $1 - 2.718^{-0.05}$, or $P_{T=5}$ = 0.049, or slightly less than 5% probability. (This is merely an illustrative example.)

4.2 RISK ANALYSIS RESULTS

4.2.1 System-level fragility

Grid control facility. Under current conditions, it is estimated that the grid control facility is more likely than not to fail at least temporarily when subjected to *PGA* is excess of 0.60g, primarily because of equipment damage. In this facility, it makes little difference if damage to identical components located in the same location and orientation is assumed to be perfectly correlated or uncorrelated. The single most effective mitigation measure for this facility would be to anchor the electric fuel pump and day tank, and to provide spacers for the starter motor batteries. If these components were to be made seismically rugged, doing so would increase to 0.71g the *PGA* at which failure is more likely than not. By strengthening all components identified here as possible weak links, the *PGA* associated with 50% failure probability is increased to 0.80g. See Figure 4-1a.

Data processing facility. Under current conditions, it is estimated that the data processing facility is more likely than not to fail at least temporarily when subjected to *PGA* is excess of 0.25g, primarily because of equipment damage. If damage to identical components in the same location and orientation are assumed to uncorrelated, failure is more likely. The single

most effective mitigation measure for this facility would be to mitigate the tape silos, either by seismic restraint (if that is possible for these pieces of equipment) or by relocating them at a seismically stable location. If the tape silos could be made seismically rugged, doing so would increase to 0.33g the *PGA* at which failure is more likely than not. By strengthening all components identified here as possible weak links, the *PGA* associated with 50% failure probability is increased to 0.85g. Note that we have assumed that damage to the tape silos is perfectly correlated—if one fails, all fail, which we deem reasonable given their proximity. See Figure 4-1b.

Backup data facility. Under current conditions, it is estimated that the backup data facility is more likely than not to fail at least temporarily when subjected to *PGA* is excess of 0.30g, primarily because of equipment damage. As with the data processing facility, if damage to identical components in the same location and orientation is assumed to be uncorrelated, failure probability is higher. As with the data processing facility, if the tape silos could be made seismically rugged, doing so would increase to 0.45g the *PGA* at which failure is more likely than not. By strengthening all components identified here as "possible weak links," the *PGA* associated with 50% failure probability is increased to 0.85g. See Figure 4-1c.

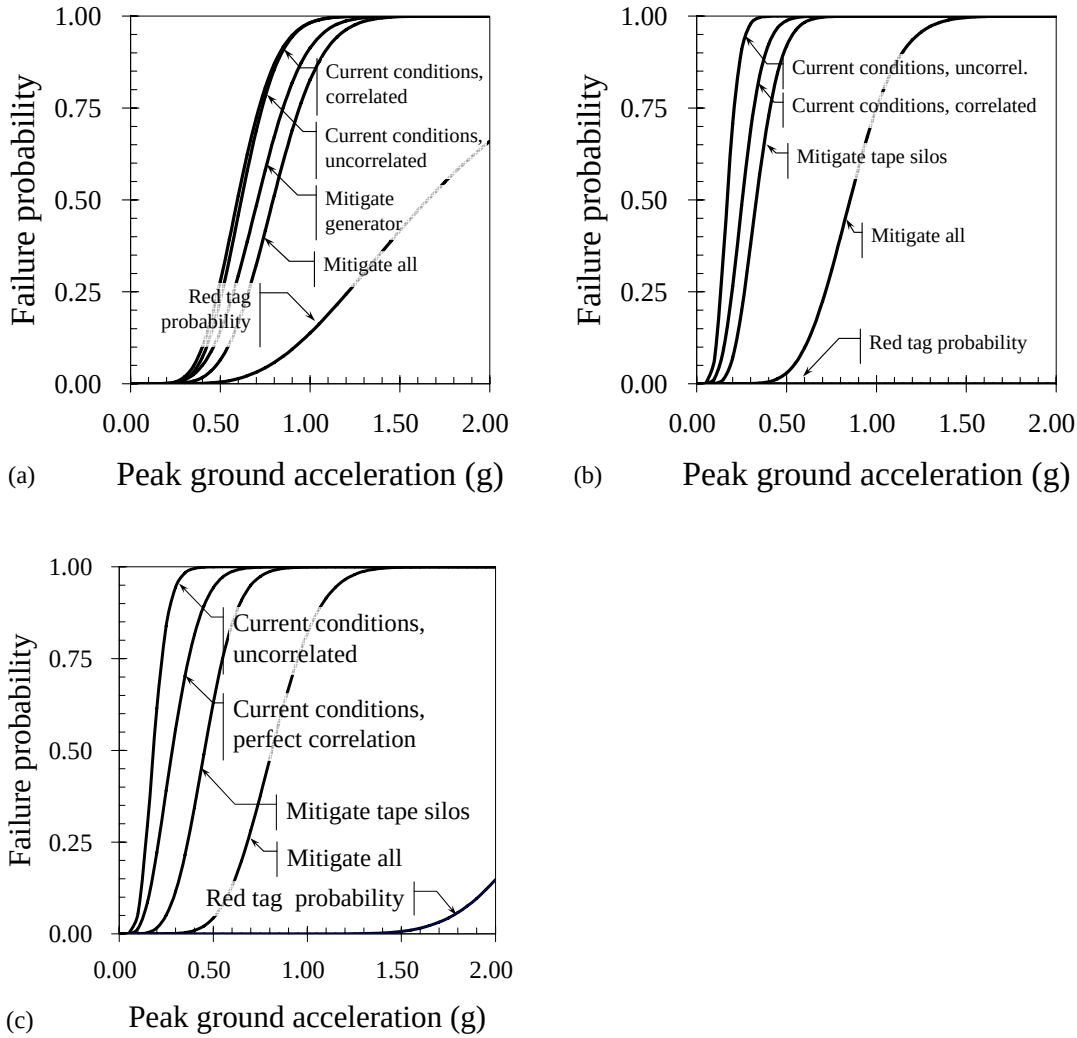

Figure 4-1. Estimated overall fragility of individual facilities: (a) grid control facility, (b) data processing facility, and (c) backup data facility.

4.2.2 System failure probability

The probability that both the data processing facility and the backup facility will fail in a single earthquake in the next five years is estimated to be 0.4%. The probability that both the grid control facility and the backup facility will fail in a single earthquake in the next five years is estimated to be 0.05%. By remediating all items notes in Section 3 as “possible weak links,” the probabilities of either failure are estimated to be less than 1 in 100,000. Details are shown in Table 4-1.

Table 4-1. Risk analysis results: estimated failure probability within the next 5 years

Conditions	Data processing facility	Backup data facility	Grid control facility	Data processing and backup	Grid control and backup
Current	0.055	0.032	0.008	0.004	0.0005
Remediate all	0.002	0.001	0.003	2×10^{-6}	2×10^{-6}

5 CONCLUSIONS AND RECOMMENDATIONS

Two primary computer centers and a backup facility that are operated by a southern California utility were examined to estimate the probability that a single earthquake to interrupt critical operations. A seismic hazard analysis was performed to identify all the earthquakes considered by the US Geological Survey that could affect all three facilities, and to estimate their occurrence probability and shaking at each facility. A systems analysis was performed to estimate the probability in each earthquake of either the buildings being rendered unsafe to occupy (red-tagged) or for their equipment to be damaged so that the facilities become inoperative.

All three facilities are structurally robust, and most equipment has been seismically anchored or braced. A few key components lack seismic restraint, however. Their potential for failure produces a possibly unacceptable level of risk, on the order of 0.1% probability that grid control or data processing would fail in a single earthquake in the next five years. Remediating these remaining components is estimated to reduce risk to below 1 in 100,000. We recommend the utility consider doing so. These conclusions are based on the assumption that equipment anchorage observed in the facilities has been tested to ensure proper installation. If the utility has not done so already, we recommend that such tests be considered as well.

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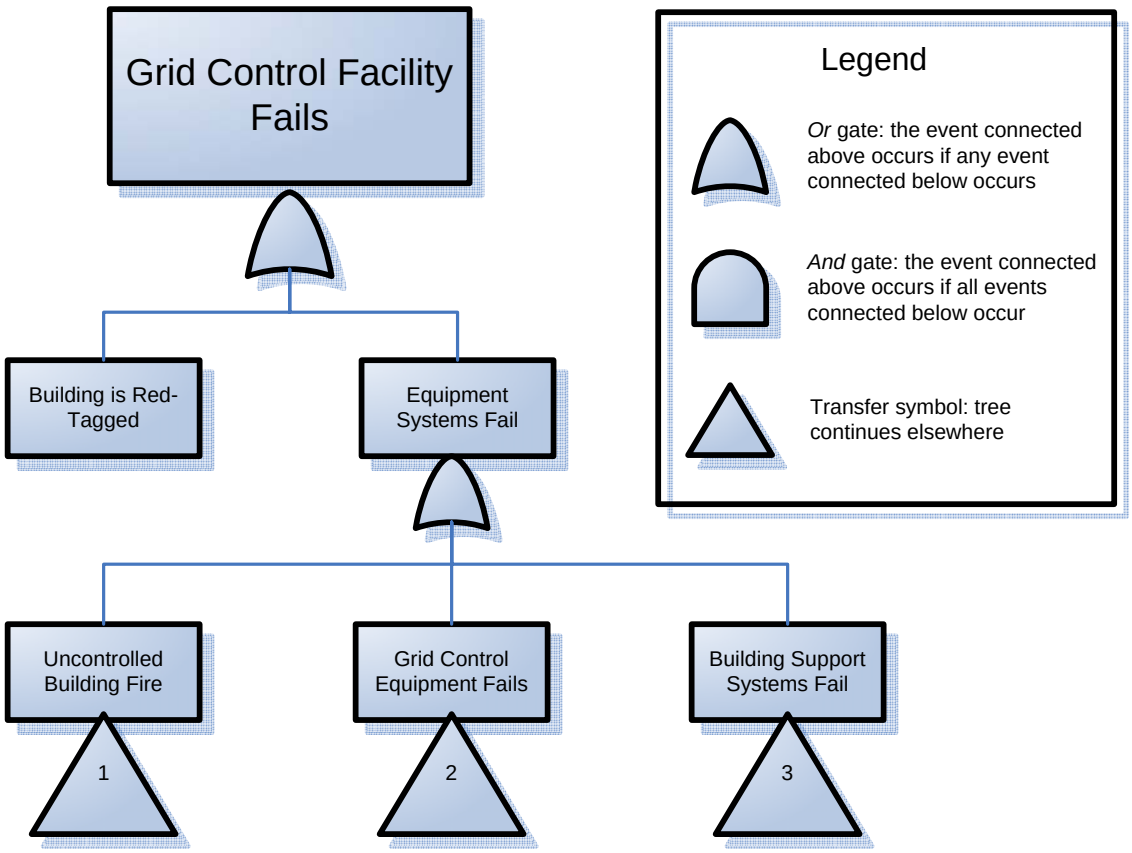
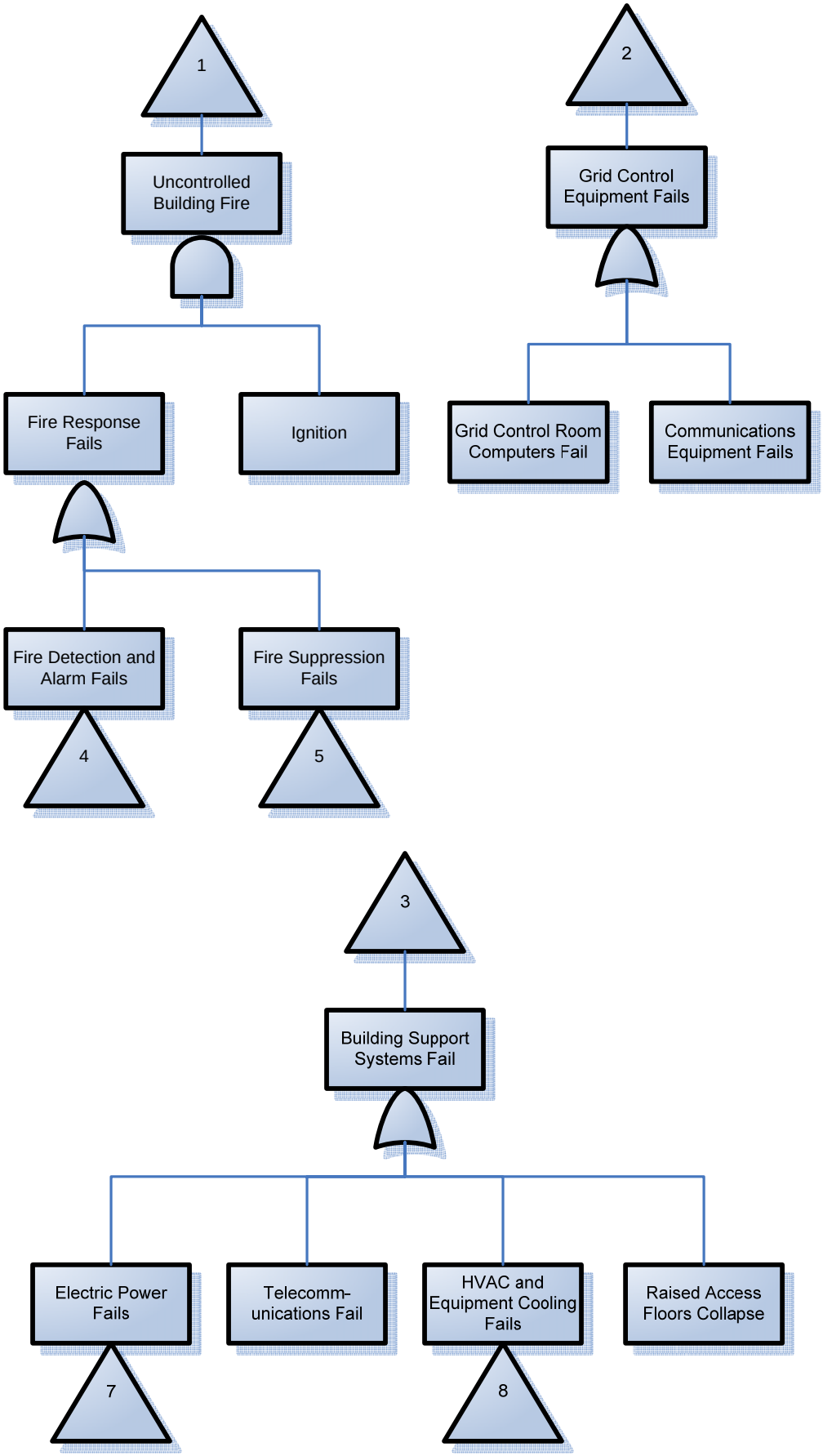
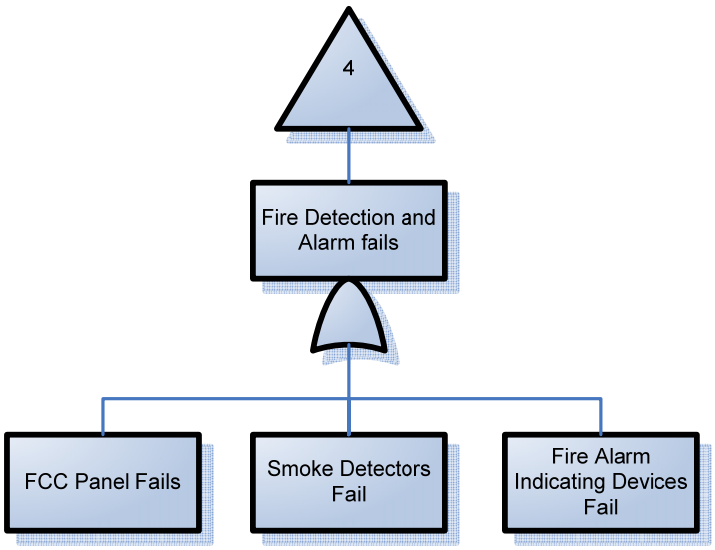
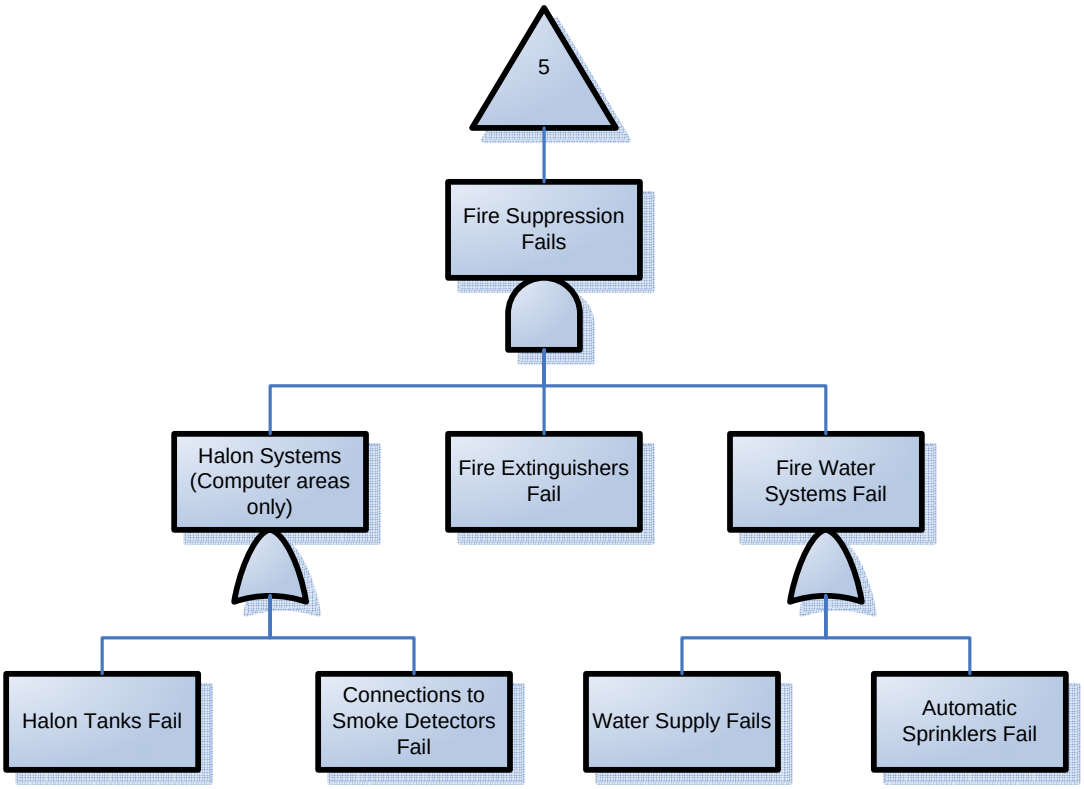
APPENDIX A:

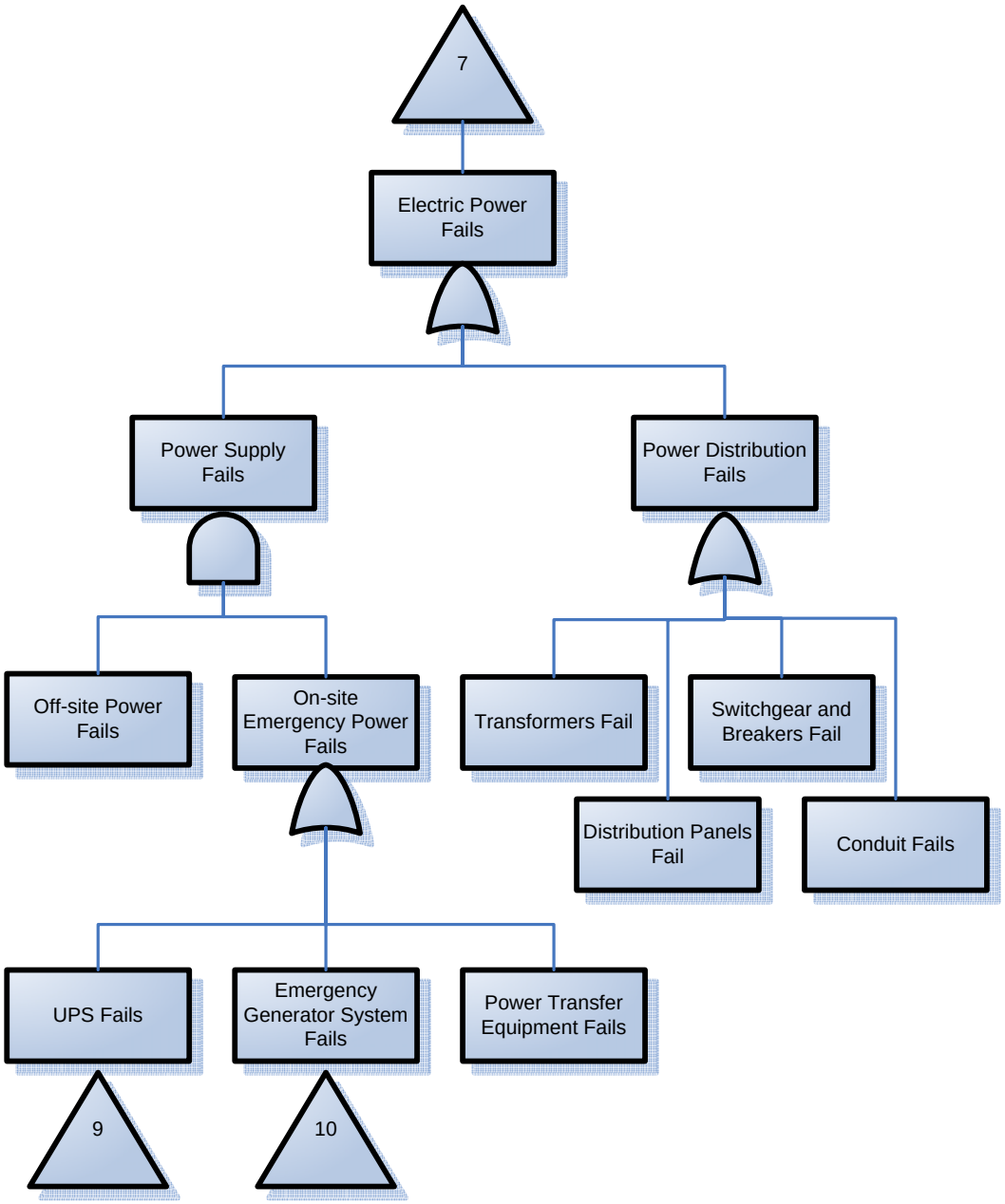
GRID CONTROL FACILITY FAULT TREE

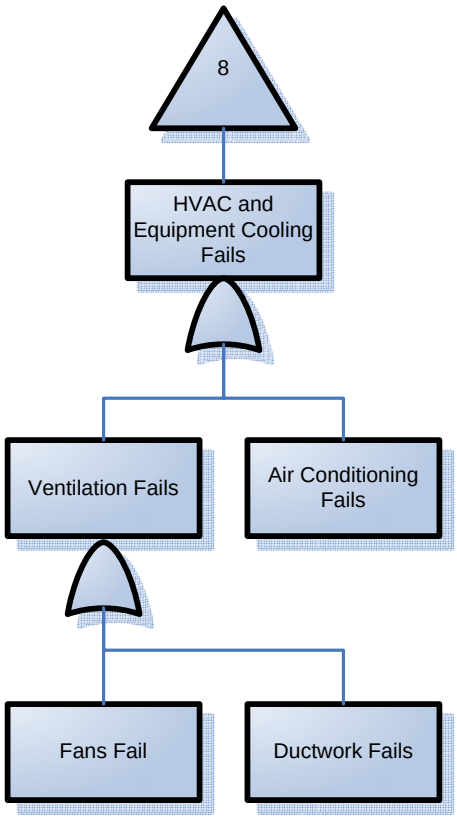


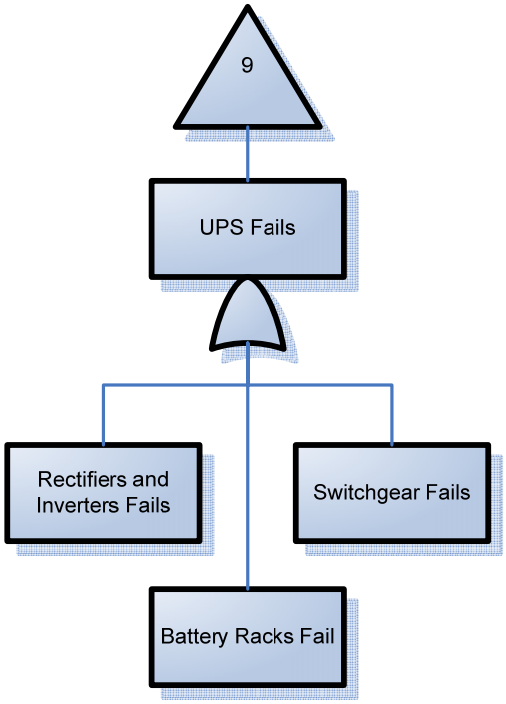

1
Uncontrolled Building Fire
Fire Response Fails
Ignition
Fire Detection and Alarm Fails
Fire Suppression Fails
4
5
2
Grid Control Equipment Fails
Grid Control Room Computers Fail
Communications Equipment Fails
3
Building Support Systems Fail
Electric Power Fails
Telecomm-unications Fail
HVAC and Equipment Cooling Fails
Raised Access Floors Collapse
7
8

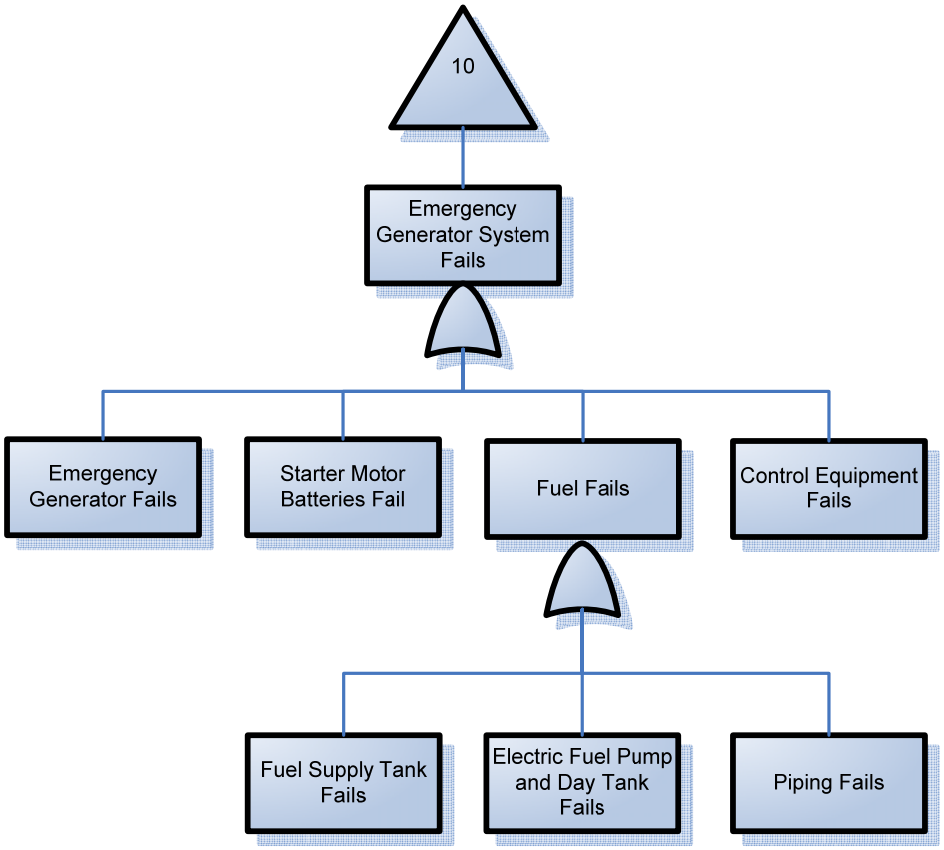

4
Fire Detection and Alarm fails
FCC Panel Fails
Smoke Detectors Fail
Fire Alarm Indicating Devices Fail


5
Fire Suppression Fails
Halon Systems (Computer areas only)
Fire Extinguishers Fail
Fire Water Systems Fail
Halon Tanks Fail
Connections to Smoke Detectors Fail
Water Supply Fails
Automatic Sprinklers Fail


7
Electric Power Fails
Power Supply Fails
Power Distribution Fails
Off-site Power Fails
On-site Emergency Power Fails
Transformers Fail
Switchgear and Breakers Fail
Distribution Panels Fail
Conduit Fails
UPS Fails
Emergency Generator System Fails
Power Transfer Equipment Fails
9
10


8
HVAC and Equipment Cooling Fails
Ventilation Fails
Air Conditioning Fails
Fans Fail
Ductwork Fails


9
UPS Fails
Rectifiers and Inverters Fails
Switchgear Fails
Battery Racks Fail


10
Emergency Generator System Fails
Emergency Generator Fails
Starter Motor Batteries Fail
Fuel Fails
Control Equipment Fails
Fuel Supply Tank Fails
Electric Fuel Pump and Day Tank Fails
Piping Fails

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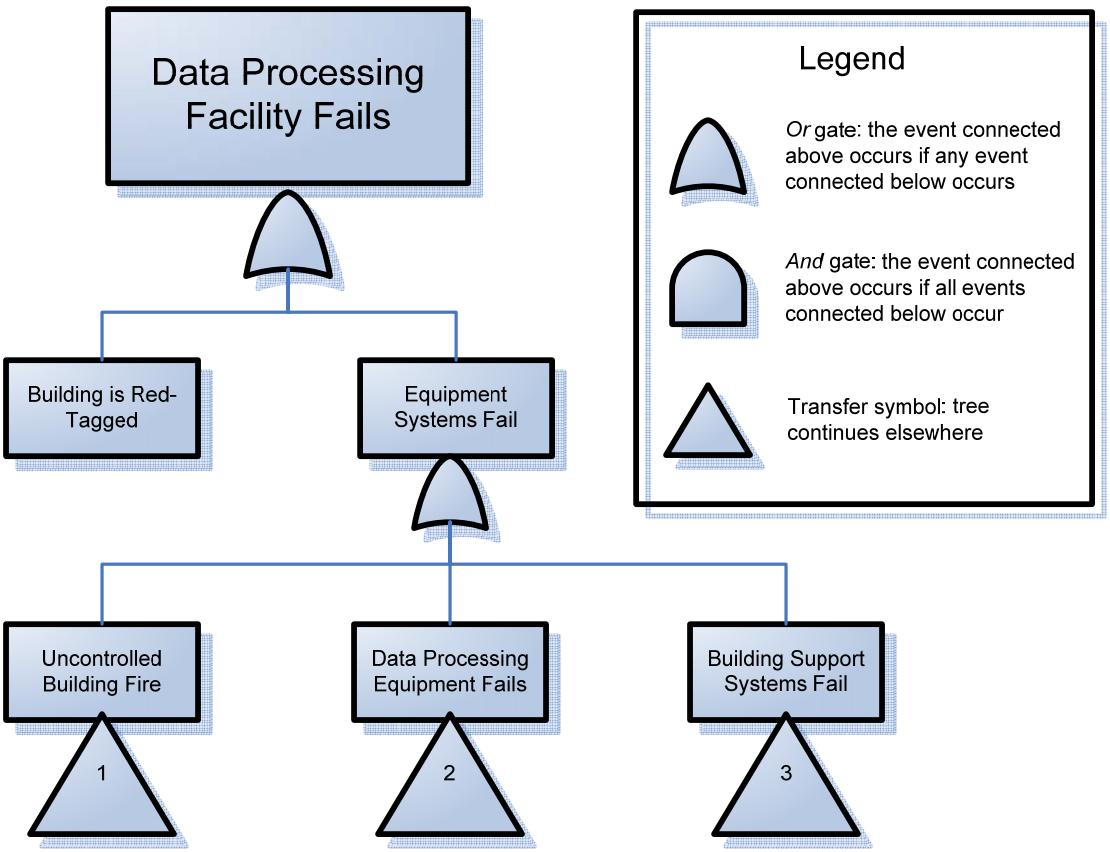
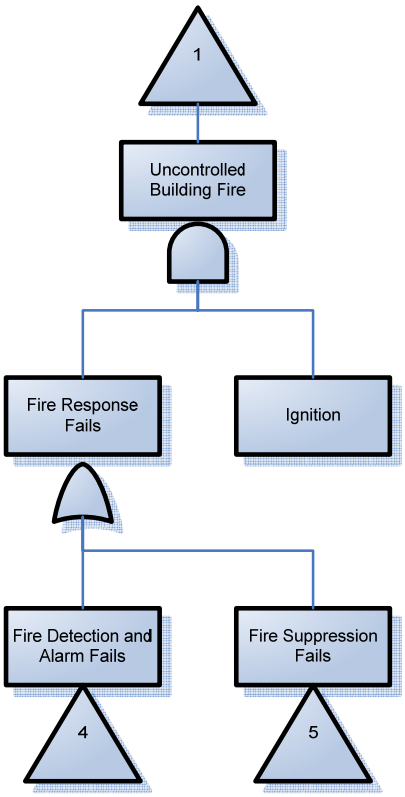
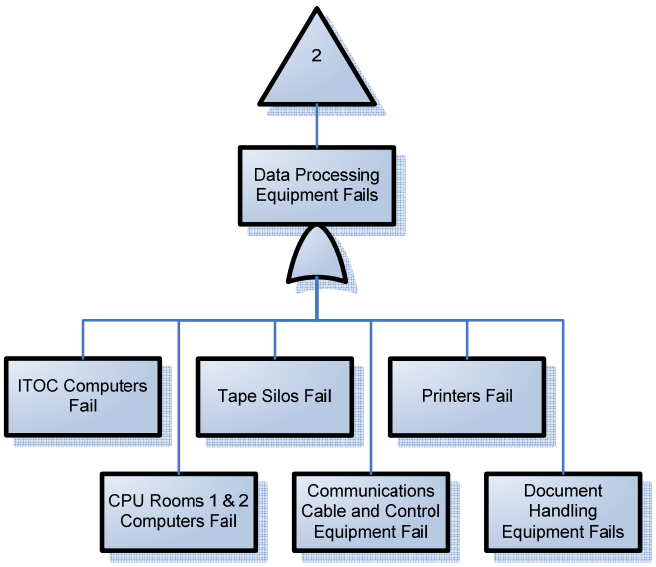
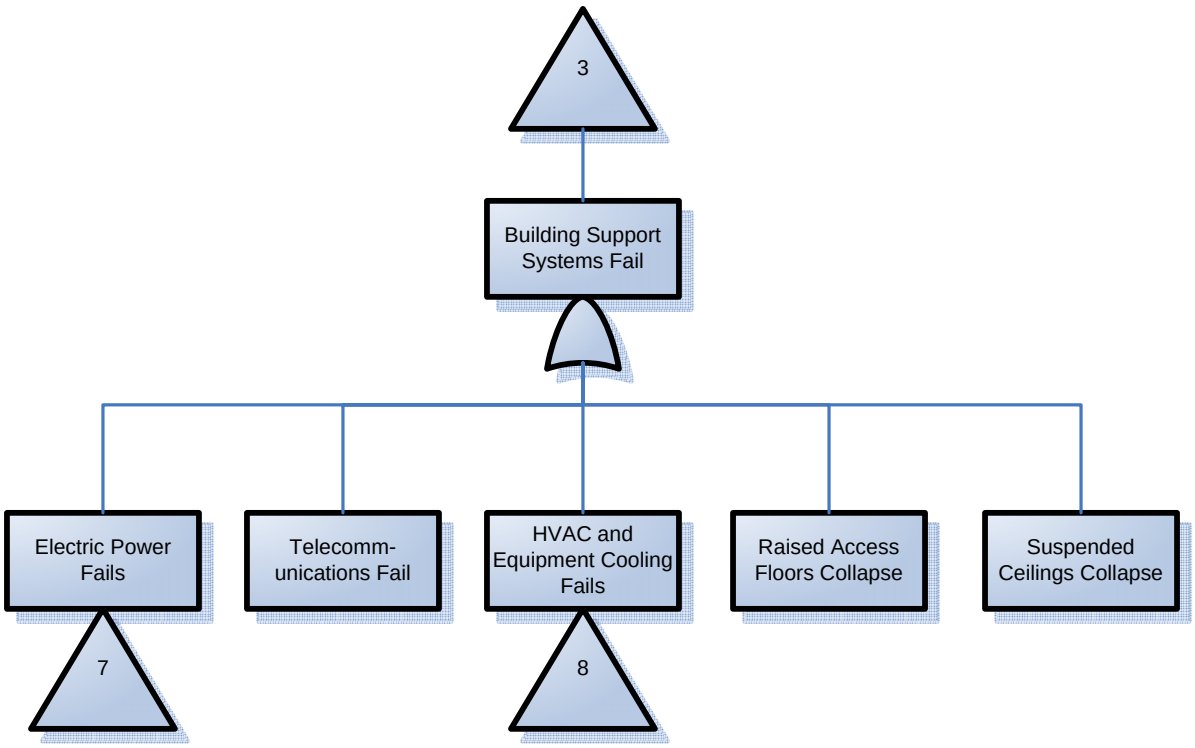
APPENDIX B:

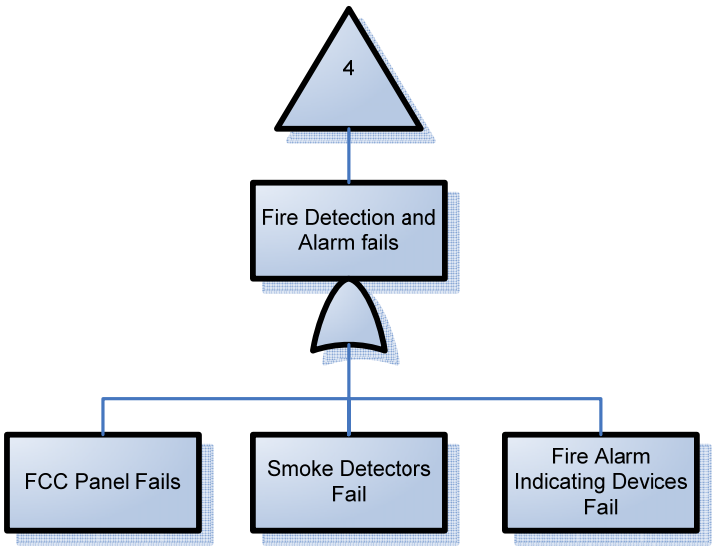
DATA PROCESSING FACILITY FAULT TREE

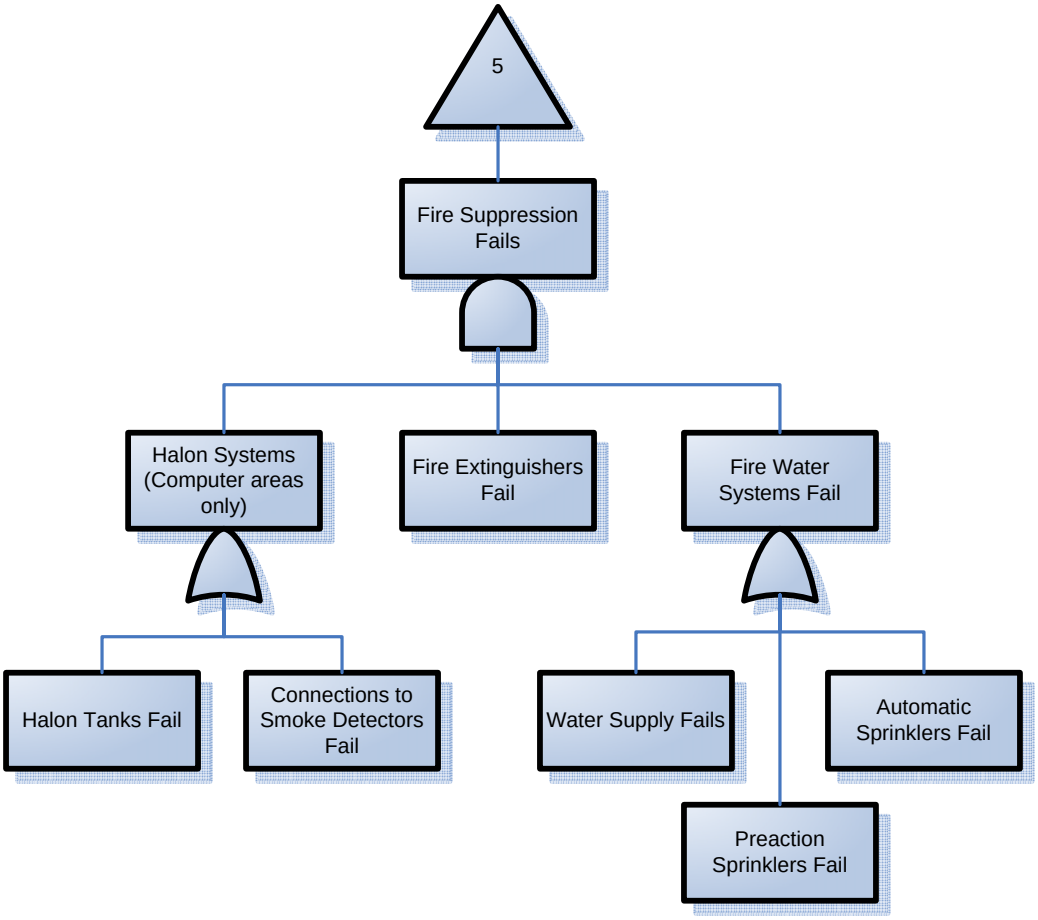


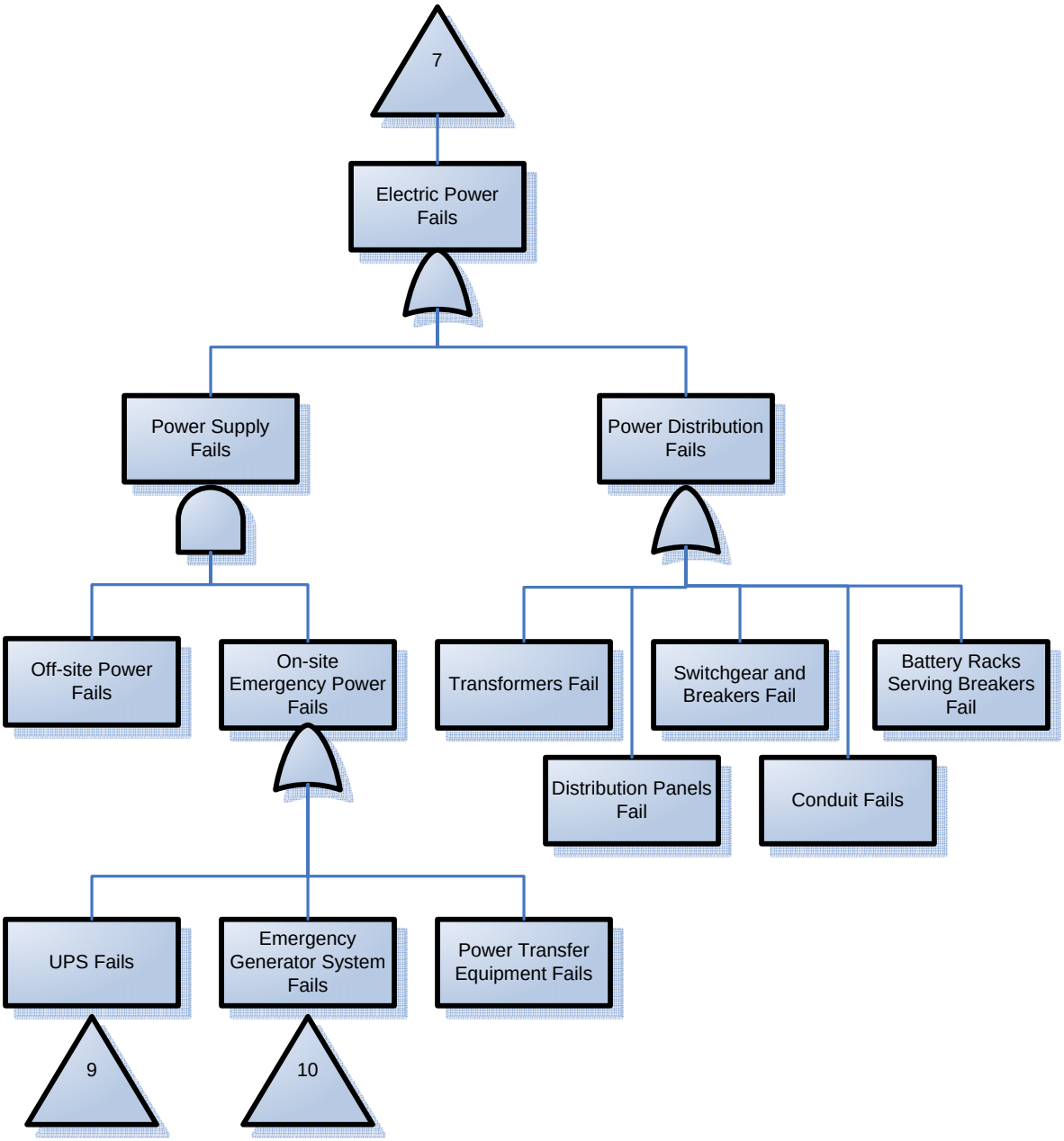

1
Uncontrolled Building Fire
Fire Response Fails
Ignition
Fire Detection and Alarm Fails
Fire Suppression Fails
4
5

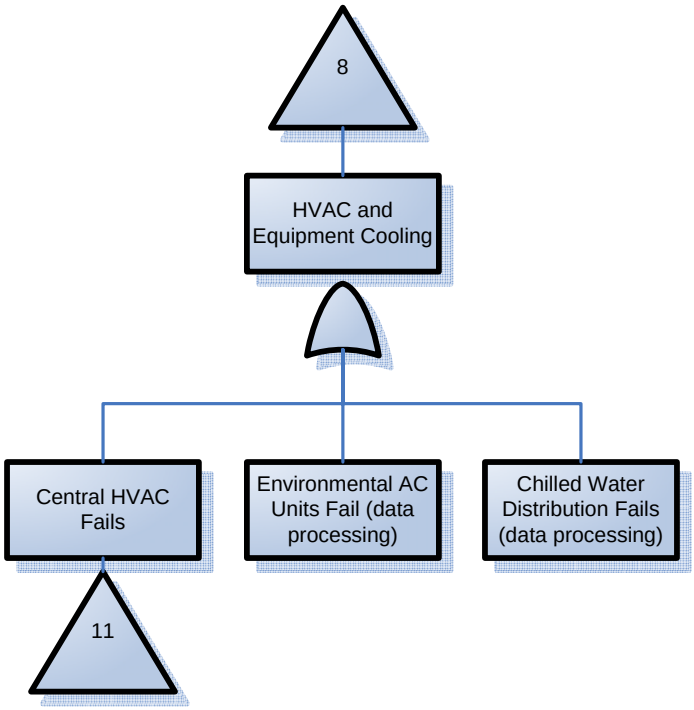

2
Data Processing Equipment Fails
ITOC Computers Fail
Tape Silos Fail
Printers Fail
CPU Rooms 1 & 2 Computers Fail
Communications Cable and Control Equipment Fail
Document Handling Equipment Fails

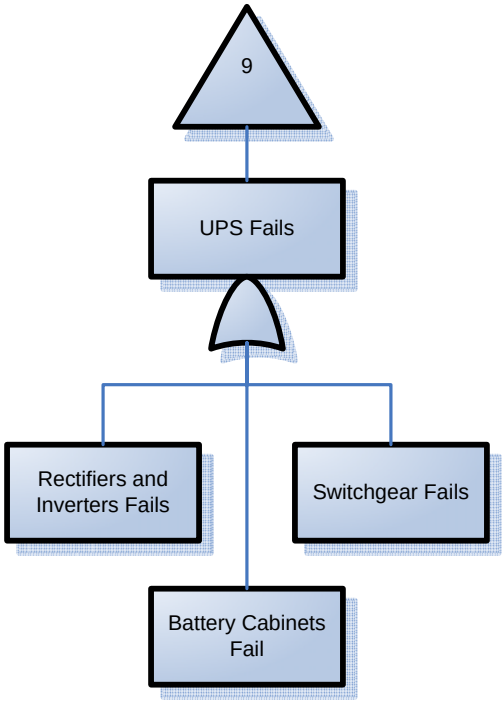

3
Building Support Systems Fail
Electric Power Fails
Telecomm-unications Fail
HVAC and Equipment Cooling Fails
Raised Access Floors Collapse
Suspended Ceilings Collapse
7
8

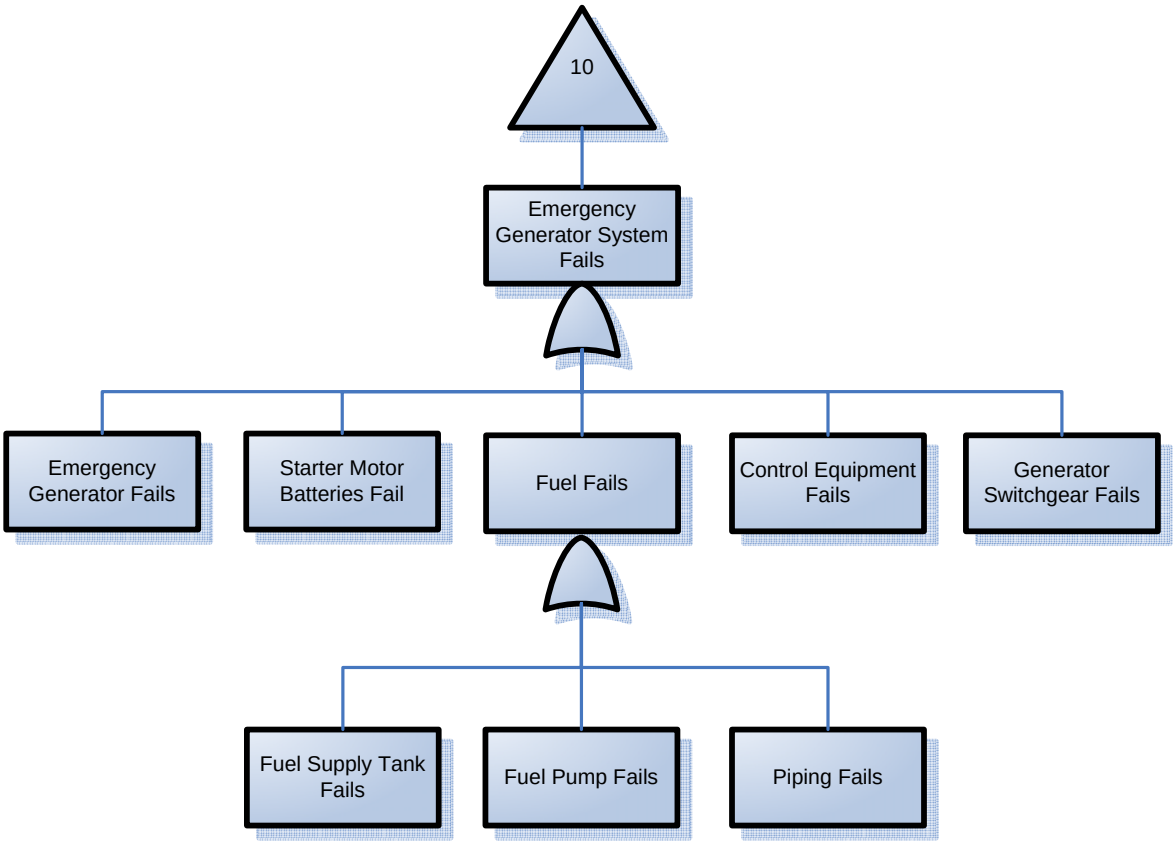

4
Fire Detection and Alarm fails
FCC Panel Fails
Smoke Detectors Fail
Fire Alarm Indicating Devices Fail

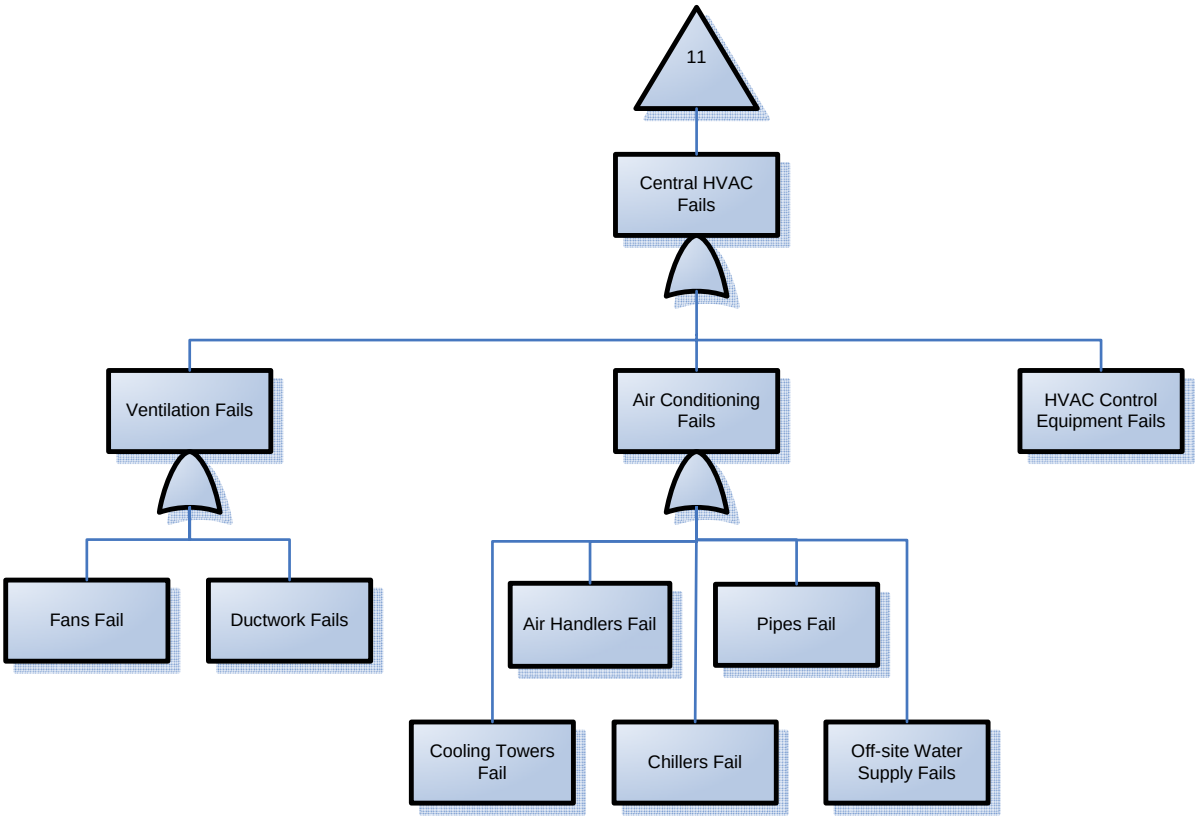

5
Fire Suppression Fails
Halon Systems (Computer areas only)
Fire Extinguishers Fail
Fire Water Systems Fail
Halon Tanks Fail
Connections to Smoke Detectors Fail
Water Supply Fails
Automatic Sprinklers Fail
Preaction Sprinklers Fail


7
Electric Power Fails
Power Supply Fails
Power Distribution Fails
Off-site Power Fails
On-site Emergency Power Fails
Transformers Fail
Switchgear and Breakers Fail
Battery Racks Serving Breakers Fail
Distribution Panels Fail
Conduit Fails
UPS Fails
Emergency Generator System Fails
Power Transfer Equipment Fails
9
10


8
HVAC and Equipment Cooling
Central HVAC Fails
Environmental AC Units Fail (data processing)
Chilled Water Distribution Fails (data processing)
11

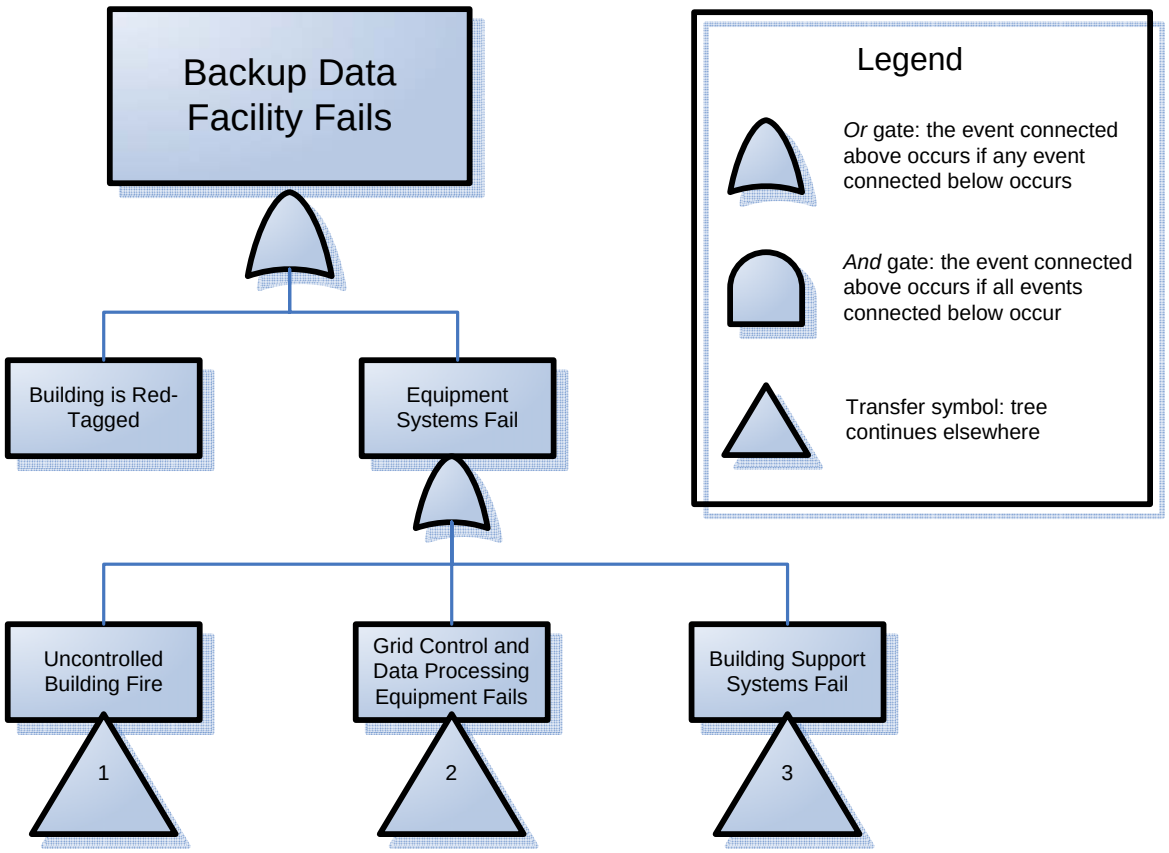

9
UPS Fails
Rectifiers and Inverters Fails
Switchgear Fails
Battery Cabinets Fail

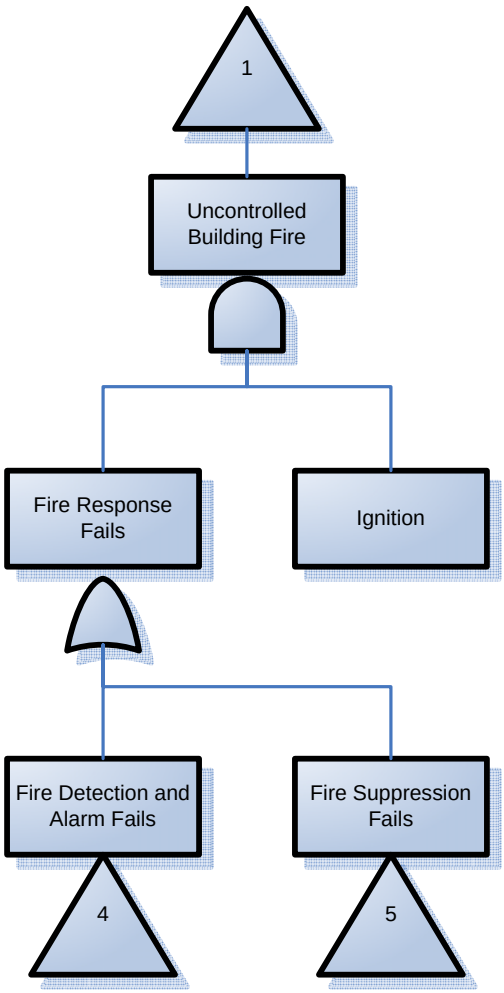

10
Emergency Generator System Fails
Emergency Generator Fails
Starter Motor Batteries Fail
Fuel Fails
Control Equipment Fails
Generator Switchgear Fails
Fuel Supply Tank Fails
Fuel Pump Fails
Piping Fails

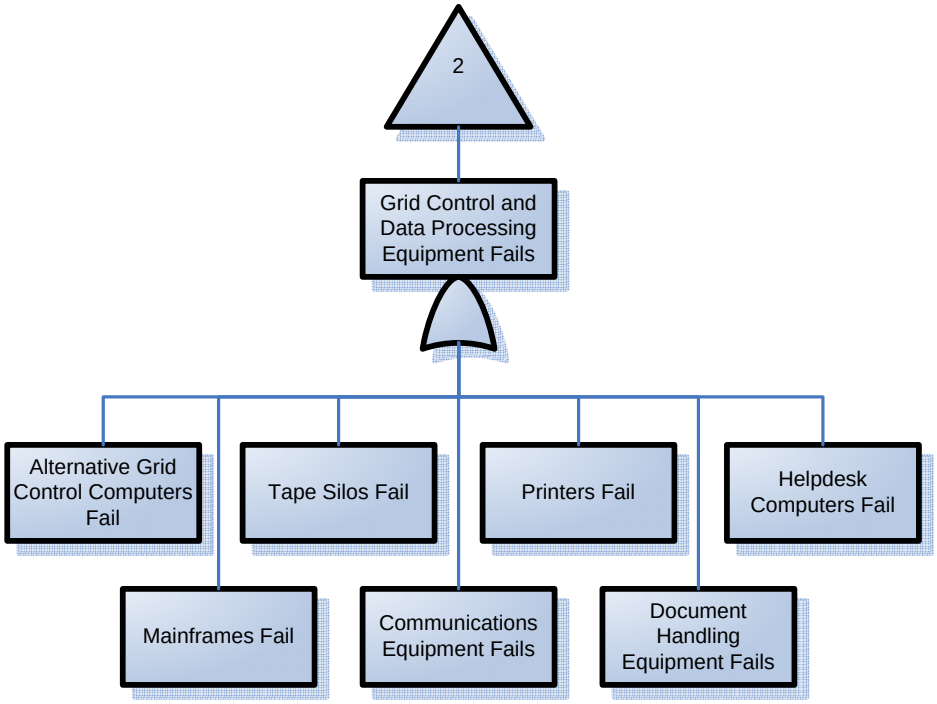

11
Central HVAC Fails
Ventilation Fails
Air Conditioning Fails
HVAC Control Equipment Fails
Fans Fail
Ductwork Fails
Air Handlers Fail
Pipes Fail
Cooling Towers Fail
Chillers Fail
Off-site Water Supply Fails

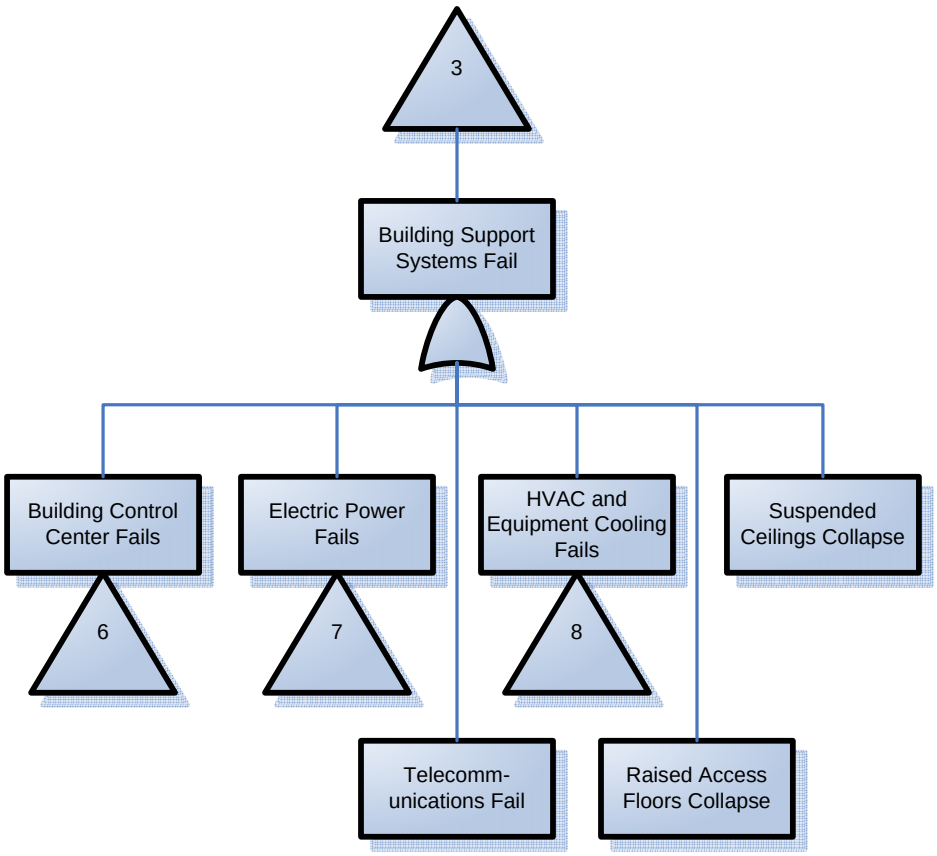
APPENDIX C:

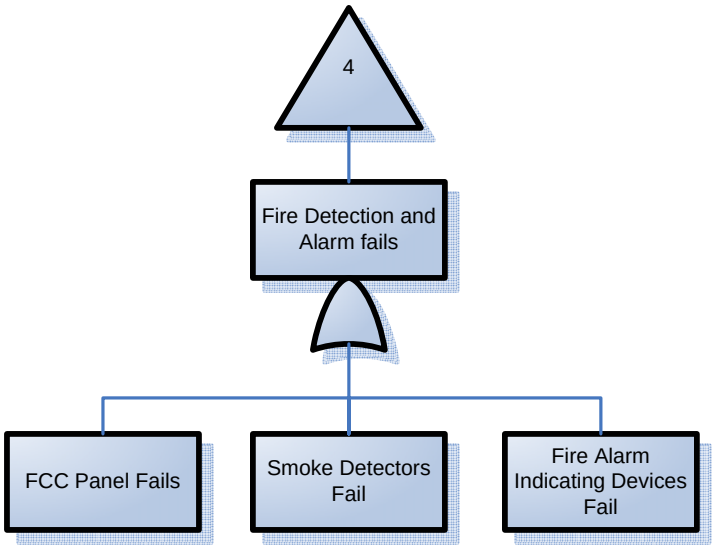
BACKUP DATA FACILITY FAULT TREE

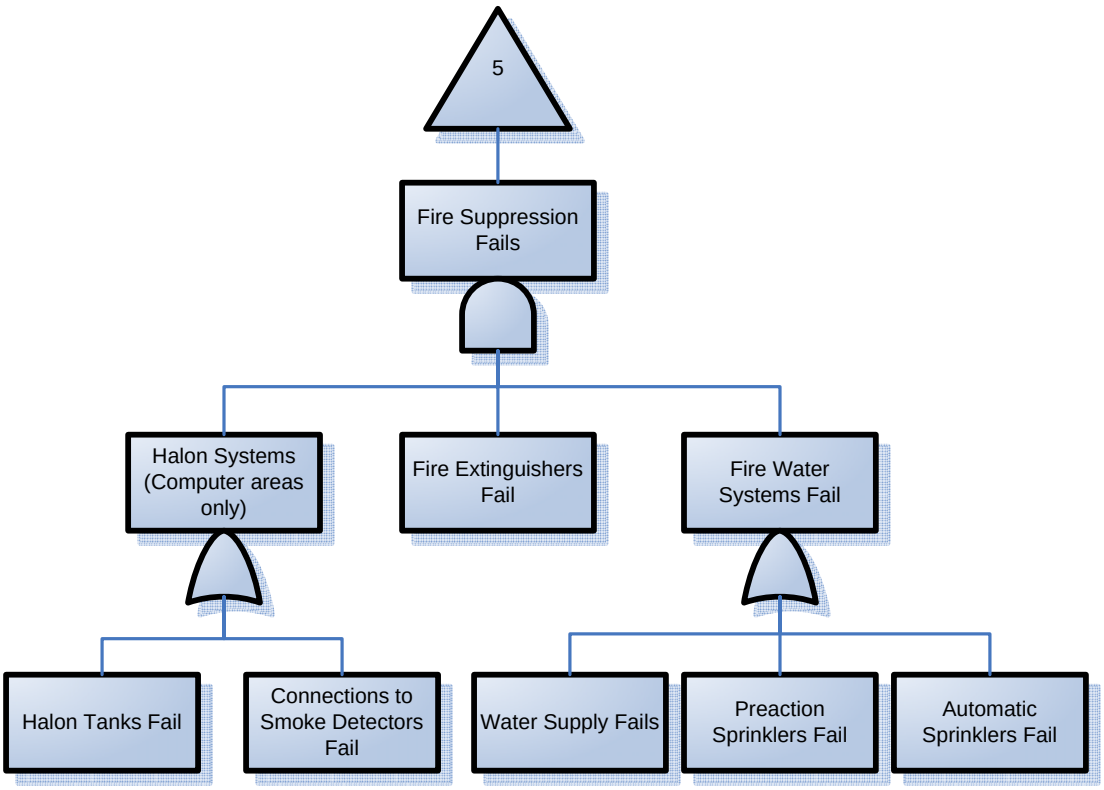


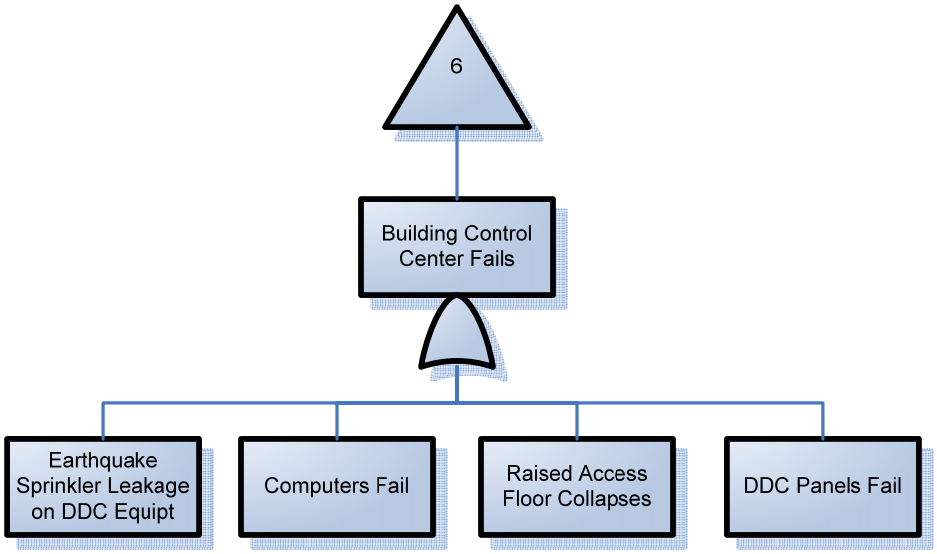

1
Uncontrolled Building Fire
Fire Response Fails
Ignition
Fire Detection and Alarm Fails
Fire Suppression Fails
4
5

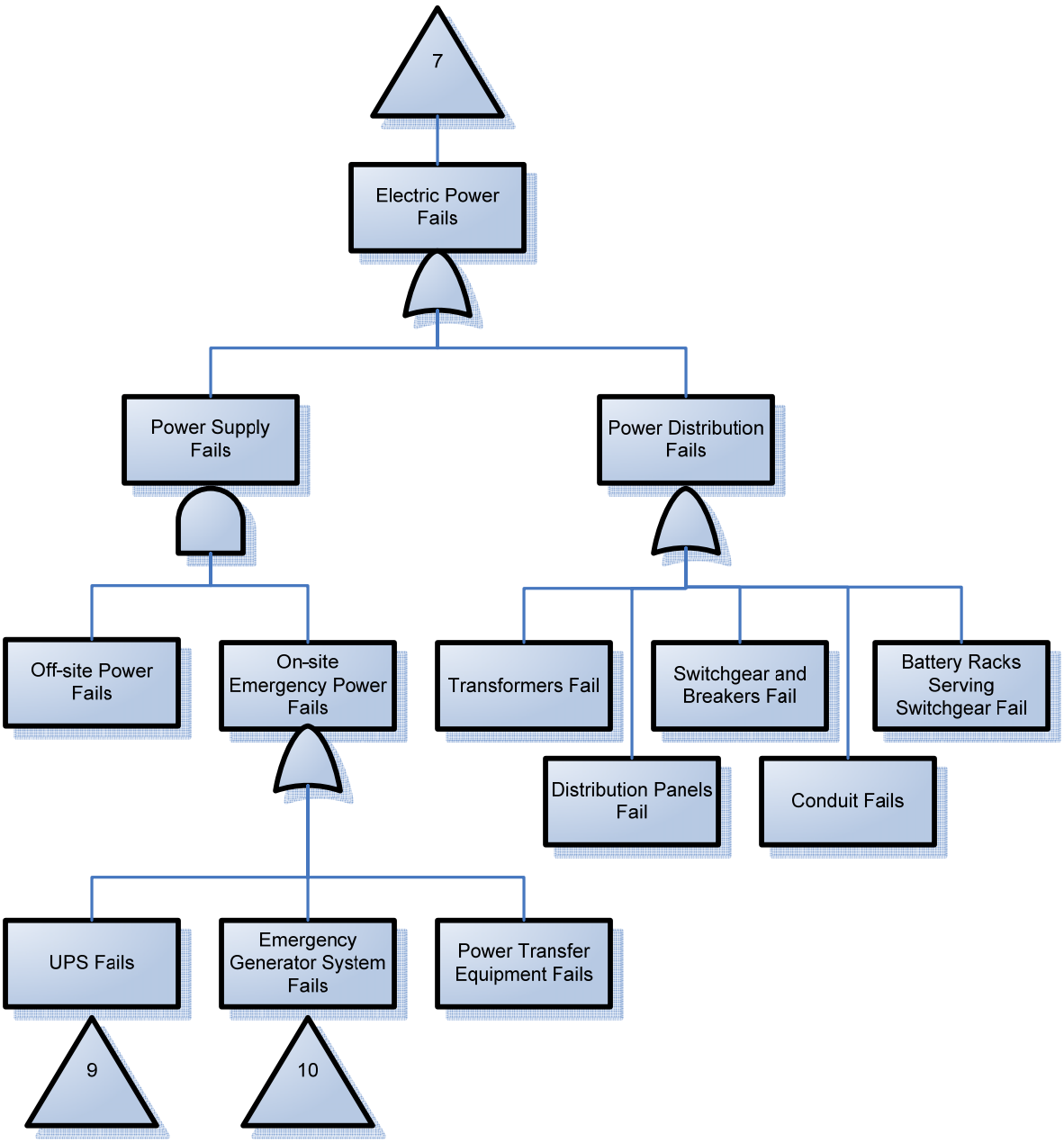

2
Grid Control and Data Processing Equipment Fails
Alternative Grid Control Computers Fail
Tape Silos Fail
Printers Fail
Helpdesk Computers Fail
Mainframes Fail
Communications Equipment Fails
Document Handling Equipment Fails

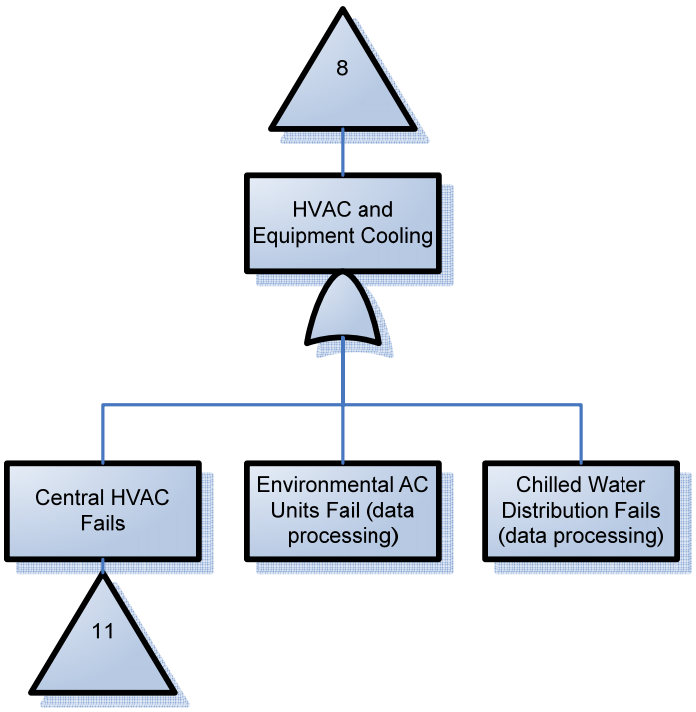

3
Building Support Systems Fail
Building Control Center Fails
Electric Power Fails
HVAC and Equipment Cooling Fails
Suspended Ceilings Collapse
6
7
8
Telecomm-unications Fail
Raised Access Floors Collapse

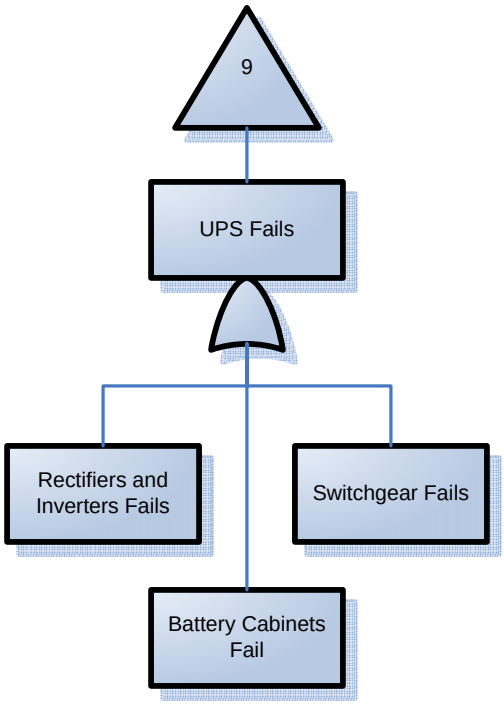

4
Fire Detection and Alarm fails
FCC Panel Fails
Smoke Detectors Fail
Fire Alarm Indicating Devices Fail

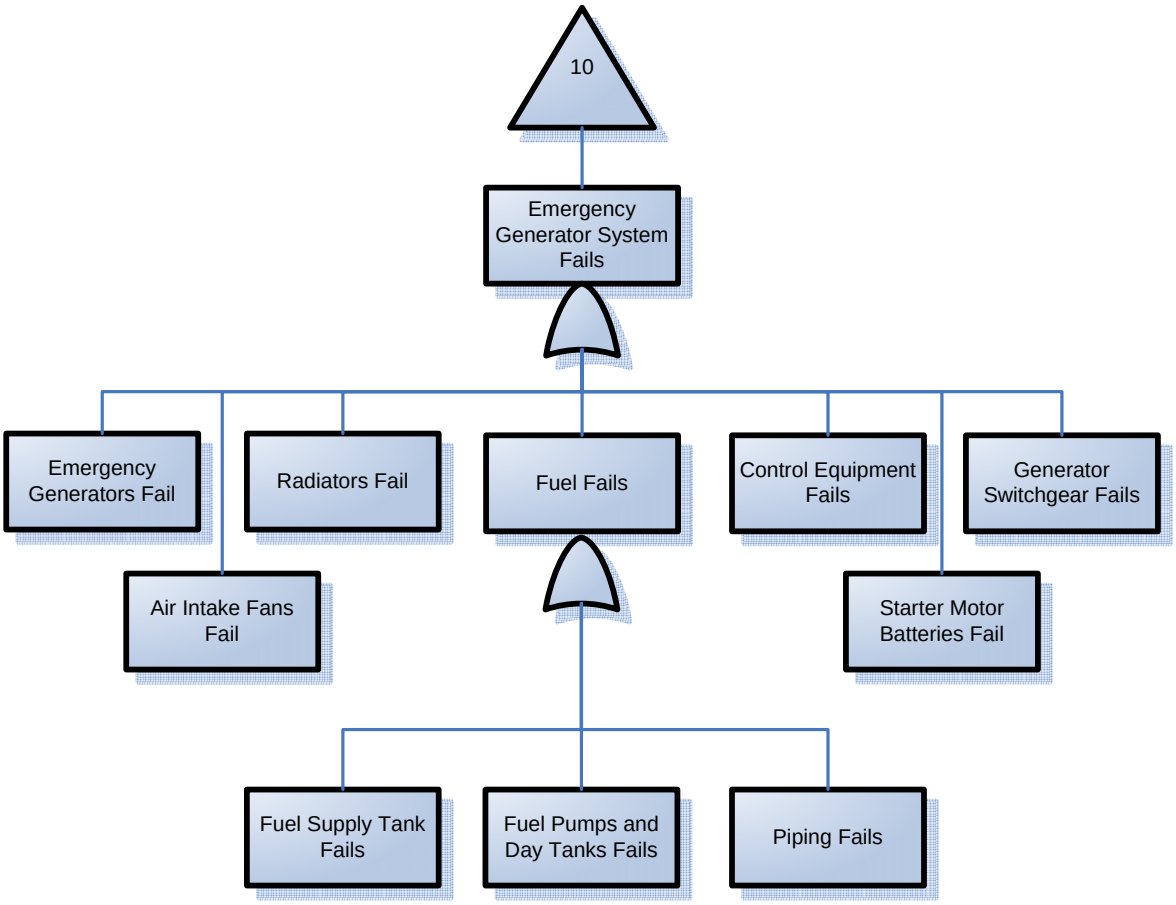

5
Fire Suppression Fails
Halon Systems (Computer areas only)
Fire Extinguishers Fail
Fire Water Systems Fail
Halon Tanks Fail
Connections to Smoke Detectors Fail
Water Supply Fails
Preaction Sprinklers Fail
Automatic Sprinklers Fail

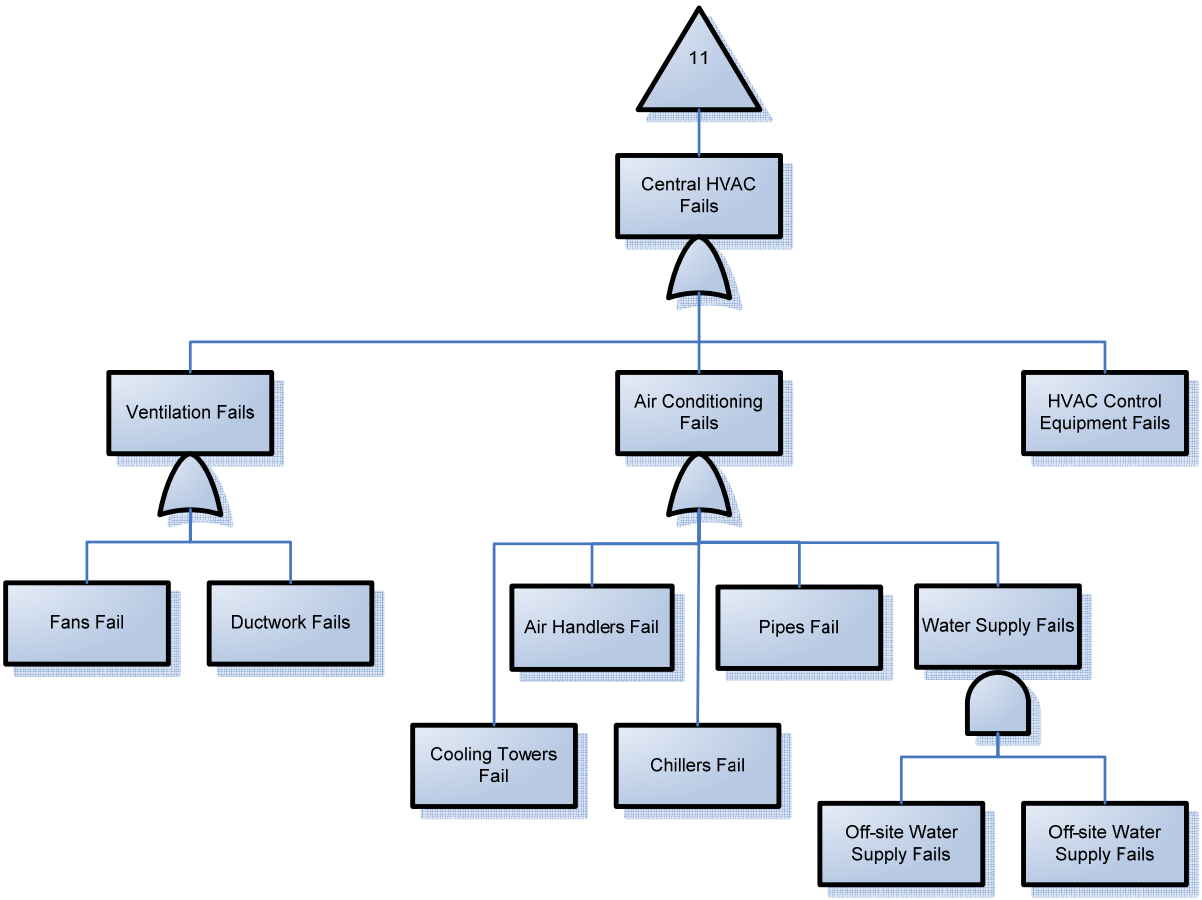

6
Building Control Center Fails
Earthquake Sprinkler Leakage on DDC Equipt
Computers Fail
Raised Access Floor Collapses
DDC Panels Fail


7
Electric Power Fails
Power Supply Fails
Power Distribution Fails
Off-site Power Fails
On-site Emergency Power Fails
Transformers Fail
Switchgear and Breakers Fail
Battery Racks Serving Switchgear Fail
Distribution Panels Fail
Conduit Fails
UPS Fails
Emergency Generator System Fails
Power Transfer Equipment Fails
9
10


8
HVAC and Equipment Cooling
Central HVAC Fails
Environmental AC Units Fail (data processing)
Chilled Water Distribution Fails (data processing)
11


9
UPS Fails
Rectifiers and Inverters Fails
Switchgear Fails
Battery Cabinets Fail


10
Emergency Generator System Fails
Emergency Generators Fail
Radiators Fail
Fuel Fails
Control Equipment Fails
Generator Switchgear Fails
Air Intake Fans Fail
Starter Motor Batteries Fail
Fuel Supply Tank Fails
Fuel Pumps and Day Tanks Fails
Piping Fails


11
Central HVAC Fails
Ventilation Fails
Air Conditioning Fails
HVAC Control Equipment Fails
Fans Fail
Ductwork Fails
Air Handlers Fail
Pipes Fail
Water Supply Fails
Cooling Towers Fail
Chillers Fail
Off-site Water Supply Fails
Off-site Water Supply Fails

APPENDIX D:
DEFENSIBILITY AND STATE-OF-THE-ART ASPECTS OF THE STUDY

D.1 HAZARD ANALYSIS

The seismic hazard analysis was performed by experienced USGS personnel using the same earthquake rupture forecast and seismic attenuation relationships employed in the 2002 National Seismic Hazard Maps, the official standard of the US Geological Survey. Furthermore, site soil information for the three facilities examined here was taken from an authoritative map created by the California Geological Survey for the purposes of seismic hazard analysis.

Despite the authoritative and well established nature of the seismic hazard analysis, the present analysis contains state-of-the-art features as well. State-of-the-art software developed by the US Geological Survey and the Southern California Earthquake Consortium was used to create these estimates of shaking intensity. The software is called OpenSHA; the interested reader is referred to the OpenSHA web page, at www.opensha.org, and to a publication by Field et al. (2005). One new crucial component was developed by the USGS explicitly for projects such as the present one; it is used here in practice for the first time. This component, entitled IM_EventSetCalc.jar, is a Java application that produces a database of intensity measure levels for an arbitrary number of intensity measure types, intensity measure relationships, and sites of interest. In addition to these two features—the use of OpenSHA and the first practical use of a new component for OpenSHA—a third way in which the hazard analysis is state of the art is in the separation of inter- and intra-event variability of ground motion.

Note that the new OpenSHA component IM_EventSetCalc.jar was not developed using the funds or under the contract of the present project. There is no contractual relationship between the USGS and Caltech or between the research sponsor and the USGS for purposes of the present project or the development of the new component. In fact, no other software was created for this project. USGS personnel checked the results of the new software's calculations to ensure that the software was working properly and agreed with independent analysis. (Specifically, the calculations were validated against the 2002 USGS National Seismic Hazard Map rock-site calculations as in Figure 2a of Field et al. 2005).

D.2 VULNERABILITY ANALYSIS

Fault-tree analysis is a mature and well-established methodology for estimating the probability of an undesirable outcome. The interested reader is referred, for example, to Vesely et al.'s (1981) handbook on fault tree analysis, prepared under the auspices of the Nuclear Regulatory Commission. Since at least the early 1990s, fault trees and closely related diagrams have been used to depict and assess the seismic reliability of various facilities other than nuclear power plants. The authors applied a version of fault-tree analysis to evaluating the seismic reliability of computer data centers and other facilities in National Science Foundation-sponsored research (Porter et al. 1993), and in practice used such a methodology for a large international financial institution to estimate the probability of simultaneous failure to two data centers conditioned on a few particular earthquake scenarios. In the later 1990s, Johnson et al. (1999) developed extensive empirical quantitative information useful for applying the methodology, under the sponsorship of the National Science Foundation. Noteworthy in the Johnson et al. (1999) work is that empirical fragility information is provided for a wide variety of equipment components, and that most if not all seismic installation conditions that matter to the fragility of each component are clearly elucidated and appropriate adjustments to the basic fragility function are provided for each feature.

Despite the credentials of fault-tree analysis, it seems likely that the present project ~~is one~~ is the first to use it to estimate the seismic reliability of multiple data centers while drawing on the empirical dataset provided by Johnson et al. (1999), or to apply the methodology to millions of earthquake scenarios. The present vulnerability analysis can therefore be said to be state-of-the-art in its use of fault tree analysis for data-center probabilistic seismic reliability assessment, especially with its strong empirical dataset.

The structural analyses were performed by a licensed Structural Engineer using state-of-the-art finite element software and a consensus methodology developed within the last 10 years by the American Society of Civil Engineers for the Federal Emergency Management Agency.

D.3 RISK ANALYSIS

The mathematics of the risk analysis rely on two well-established mathematical principles: the theorem of total probability and the mathematics associated with a Poisson process, whose details will not be explained here. For the benefit of the technical reader, we

note that the assumption of earthquake occurrence as a Poisson process is common and furthermore is consistent with the 2002 National Seismic Hazard Mapping project.

The state-of-the-art aspects of the seismic hazard analysis and the vulnerability analysis suggest that the overall study may be unprecedented in its rigor and sophistication: this may be the first time a fully probabilistic estimate has been performed of the simultaneous operational failure in two or more distant facilities in a single earthquake during a given planning period, considering both red-tagging and equipment damage, using millions of scenario earthquakes, state-of-the-art fault tree analysis, and broad empirical basis for the equipment fragility.

APPENDIX E:
LIMITATIONS

6.1 LIMITATIONS

Sampling. The quantitative analysis relies on a sampling of equipment installation conditions at the three facilities. While fairly extensive, the sampling was not exhaustive. We did not inventory every piece of critical equipment at each facility, and document and assess the adequacy of the seismic restraint of each. Such an effort would have exceeded the time constraints. Instead, to generalize from the samples examined, we relied on statements of the expert building engineers present at the three walkthroughs. An error resulting from this simplification could cause the system failure probabilities estimated here to be either too high or too low, depending on whether an unexamined exception were more or less seismically resistant than is assumed here. We made several hundred observations and in no case did the statements of the experts appear to be contradicted by our observations, so we submit that this reliance is justifiable and any error likely to be small. Nonetheless, given importance of this issue, an exhaustive inspection should be performed.

Some systems not examined in detail. We did not examine in detail equipment that the utility's personnel told us were not critical to the continued operation of the facility. For example, we do not treat here ordinary offices at the grid control GCF, the Distributed Systems Lab at the data processing facility, or the Tape Library Room at the backup data facility. Conceivably the equipment in these rooms could be relevant to the post-earthquake operation of the facility, perhaps as redundant replacement equipment for damaged components. However, we submit that the judgment of the utility's engineers that these rooms and equipment are non-critical is authoritative and justifiable. An error resulting from this assumption could cause the system failure probabilities estimated here to be either too high or too low. For example, if equipment in these labs could be quickly used as backup, then since we have ignored their presence our failure probability estimates are (to some perhaps limited extent) conservative, i.e., high. On the other hand, the error could go the other way: damage within these rooms could conceivably lead to damage elsewhere. For example, sprinklers could break or a fire could start in these spaces, causing sprinklers to discharge; the fire-suppression water could then leak into critical spaces and damage equipment.

No mechanical testing. Many times in this report we state that items “appeared to be anchored,” or use similar verbiage. We observed bolt heads with nuts, and assumed but did not perform mechanical tests to confirm that the bolts were installed according to the engineer’s or manufacturer’s recommendations. Conceivably a contractor could have failed to install anchors according to the engineer’s or manufacturer’s recommendations, such as without epoxy in the hole. Mechanical testing of a number of these anchors should be performed to confirm the adequacy of installation.